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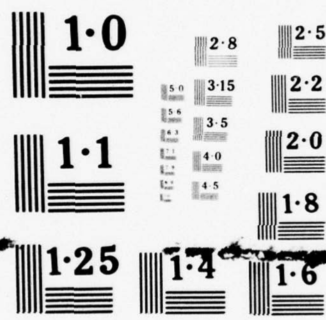

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ENGINEERING BEHAVIOR OF PAVEMENT MATERIALS: STATE OF THE ART

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16. Abstract

This report reviews the engineering behavior of pavement materials with respect to highway and aircraft loadings and environmental conditions. The materials covered are bituminous mixtures, portland cement concrete, granular materials, chemically stabilized soils, and fine-grained soils. Basic properties of each are discussed. For bituminous mixtures, emphasis is placed on the characteristics of permanent deformation, fatigue, and rheological properties and the application to pavement design of accumulative damage theory based on Miner's hypothesis. Discussions are presented on the development of fatigue criteria from laboratory fatigue tests and design curves. For portland cement concrete, concrete strengths determined by various tests are discussed. Test procedures for determining the modulus of elasticity and Poisson's ratio are presented, together with discussion of factors affecting these values. The fatigue property of concrete and its relationship to pavement design are discussed. For granular materials, the resilient and plastic properties are discussed. Constitutive stress-strain relations proposed by many agencies are presented and compared. The relations consist of resilient, plastic, shear, and dynamic stresses and strains. Because of the highly nonlinear nature of granular materials, the validity of the superposition principle as applied to pavement design is discussed. For soil stabilization, the mechanisms of stabilization are explained, which include soil-cement, soil-lime, lime-fly ash, and lime-cement-fly ash and bituminous materials. Factors influencing engineering properties and properties of stabilized soils with respect to strength, modulus, and fatigue are discussed. For fine-grained subgrade soils, discussions also concentrate on the resilient and plastic properties. Constitutive stress-strain relations are presented with respect to resilient, static, viscoelastic, plastic, dynamic, and shear properties. The modulus of subgrade reaction used in rigid pavements and the nature of expansive soils in relation to rigid pavement design are discussed.

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PREFACE

The study described herein was jointly sponsored by the Federal Aviation Administration as a part of Inter-Agency Agreement FA73WAI-377, "New Pavement Design Methodology," and by the Office, Chief of Engineers, U. S. Army, as a part of Military Construction RDT&E Project No. 4A762719AT04, "Pavements, Soils, and Foundations."

The study was conducted by the U. S. Army Engineer Waterways Experiment Station (WES), Soils and Pavements Laboratory. Dr. Yu T. Chou, under the general supervision of Messrs. James P. Sale, Richard G. Ahlvin, Ronald L. Hutchinson, and Harry H. Ulery, Jr., was in charge of the study and, with the exception of part of Chapter 3, wrote this report. In Chapter 3, Dr. Chou wrote the sections "Fatigue of Concrete and Its Relation to Pavement Performance" and "Other Measurements." The remainder of Chapter 3 was written by Mr. Billy J. Houston under the supervision of Mr. Bryant Mather, Chief of the Concrete Laboratory, WES.

COL G. H. Hilt, CE, and COL J. L. Cannon, CE, were Directors of WES during the conduct of this study and the preparation of this report. Mr. F. R. Brown was Technical Director.

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METRIC CONVERSION FACTORS

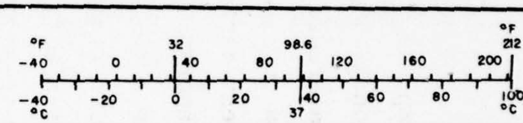
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



CHAPTER 1: INTRODUCTION

In the design and construction of airport pavements, it is important to understand the behavior of the various materials which constitute the pavement structures. It is well known that most pavement materials do not respond to traffic loadings in a linearly elastic manner as defined by the linear theory of elasticity, the backbone of most current design theories. To the contrary, laboratory tests and field observations have shown that most pavement materials respond to loadings in a nonlinear and inelastic manner. Material strength varies greatly with such environmental conditions as temperature, frost conditions, rainfall, and groundwater table variations, and also depends upon the rate and type of loading (the latter involves the load magnitude and the gear configuration of the aircraft). Pavements having the same thicknesses of component layers and constructed from similar materials, when subjected to same aircraft loadings, may perform quite differently depending on locality and environmental conditions. The response of a component layer to load and environment may also depend on the strength characteristics of overlying and underlying layers.

Attempts have been made to formulate constitutive relations of pavement materials for use in mechanistic models to predict pavement response to loadings. Limited success has been achieved, but the stress-strain relations formulated are generally accurate only for a limited range of loadings and boundary conditions. Cumulative damage theory based on Miner's hypothesis has been used in recent years to compute pavement damage from failure criteria derived from either laboratory test data or existing design curves. However, this method has been criticized because (a) laboratory testing conditions do not completely simulate field conditions, and (b) the stress-strain relations and failure criteria determined vary with testing procedures, methods, and other conditions.

In the new rational design procedure for airport pavements developed for the Federal Aviation Administration (FAA), and the Office,

Chief of Engineers, U. S. Army (OCE), mechanistic models are used to compute critical stresses and strains in pavement structures, and cumulative damage computer programs are prepared to predict damage induced by mixed aircraft loadings with consideration given to lateral distribution of different aircraft.

It is the purpose of this report to present current information on engineering behavior of pavement materials with respect to highway and aircraft loadings and environmental conditions. The pavement materials include (a) bituminous mixtures, (b) portland cement concrete, (c) granular materials, (d) chemically stabilized soils, and (e) fine-grained soils.

CHAPTER 2: BITUMINOUS MIXTURES

2.1 INTRODUCTION

In the design of bituminous mixtures for highway and airport pavements, it is important that a balance be obtained between a number of desirable mix properties. Generally, this balance necessitates a compromise in the selection of the final design bitumen content. Finn[1] listed seven pertinent mix properties which should be considered in design, and these are condensed and discussed below.

2.1.1 STABILITY

Stability has been defined as the resistance of a mix to deformation under load. The deformation implied in this definition is permanent (or plastic) deformation resulting from (a) static or nearly static loads at relatively high temperatures, or (b) rutting associated with many applications of channelized traffic. Stability is a function of (a) frictional resistance, both interparticle and intraparticle (mass viscosity), (b) cohesion, and (c) inertia. Of these, frictional resistance is the major contributor (except that cohesion can predominate at low temperatures) to resistance to deformation; and, for the high temperatures and slowly applied loads normally considered, the contribution of the interparticle friction to stability is predominant. For these circumstances, the aggregate characteristics, particularly those aspects affecting particle interaction, exert a major influence. Improper compaction, high bitumen content, or high percentage of fines tend to reduce this friction and thus permit plastic deformation to develop more readily.

2.1.2 DURABILITY

Durability of a paving mixture can be defined as its resistance to weathering, including aging, and to the abrasive action of traffic. Included in the effects of weathering are (a) changes in the characteristics of the bitumen due to such causes as volatilization, oxidation, polymerization, separation, and syneresis, and (b) changes in the mixture

due to the action of water and water vapor. To minimize the effects of weathering, experience would indicate that a high bitumen content, a dense aggregate gradation, and a well-compacted impervious mixture are required.

Sufficient bitumen must also be incorporated in the mix to provide tensile properties adequate to resist the tractive and abrasive forces of traffic. In addition, the durability characteristics of the aggregate are important in that the material must offer sufficient resistance to fracture and degradation under the forces imposed by traffic and during the construction process.

2.1.3 FLEXIBILITY

Flexibility is defined in this report only as the ability of the mixture to conform to long-term variations in base and subgrade elevations; i.e., long-term settlements resulting from consolidation of underlying soft, compressible soils and differential compaction from traffic of the underlying components of the pavement structure.

2.1.4 FATIGUE RESISTANCE

Bituminous pavements subjected to many repetitions of load may exhibit cracking of the bituminous concrete surfacing due primarily to the resilient deformations to which the material is subjected. Moreover, this cracking may be associated with little or no permanent (or irrecoverable) deformation of the pavement section. When this particular mix property is examined, however, behavior of the entire pavement section must be considered because the occurrence of fatigue cracking is not only influenced by the characteristics of the bituminous mixture, but also by the thickness and characteristics of the other components of the pavement section.

2.1.5 SKID RESISTANCE

Skid resistance is the ability of the surface of a bituminous paving mixture to provide sufficient friction so that a vehicle will be able to brake to a stop within a reasonable distance under a variety of environmental conditions. High skid resistance is generally promoted by

the same factors as high stability; i.e., comparatively low bitumen content and aggregates with rough surface textures. Suitable aggregates, in addition to possessing rough textures, must also be resistant to the polishing action of traffic. Aggregates most desirable from this standpoint are those which have minerals of different wear characteristics. Under the action of traffic, an aggregate of this type will continually have its rough surface texture renewed, thus providing the sharp coarse-grained surface necessary to develop effective contact of the tire and the pavement.

Water on the pavement is the undesirable factor in skid resistance. Recently, methods of porous friction surfacing and grooving have been used to increase pavement friction. These are discussed in conjunction with permeability in the next section.

2.1.6 PERMEABILITY

Permeability of a bituminous mixture can be simply defined as the ease with which air, water, and water vapor pass into or through the mixture. For highway pavements, it appears that mixtures with a high degree of imperviousness to air, water, and water vapor are desirable to promote long-term durability and to allow surface water to be transported to drainage facilities rather than to percolate through the pavement to the underlying components. For these conditions, then, the same factors which contribute to durability (i.e., high bitumen content, dense aggregate gradation, and "good" compaction) also insure imperviousness.

In recent years, much research has been done to develop an effective means to combat the problems of hydroplaning, skidding, and poor braking on wet pavement. For this purpose, porous friction surfaces (PFS's) have been found to be effective and economical.[2] A PFS layer has a thickness of 1/2 to 3/4 in. and consists of open-graded aggregate providing lateral internal drainage of surface water.

2.1.7 FRACTURE STRENGTH

Fracture strength is considered to be the maximum strength which a mixture exhibits when subjected to tensile forces. Fracture strength

is important when considering the application of heavy loads to pavements, particularly at low temperatures and when the underlying pavement components are comparatively weak, such as in the spring of the year in many parts of the United States. In addition, the fracture strength of the mix is important when evaluating the possibility of cracking due to volume changes, such as those which may result from temperature changes, dislocations, and movement in the underlying components of the pavement structure.

In the foregoing discussion of the seven pertinent mix properties, certain variables recur. These are bitumen content, aggregate gradation, and mix density (degree of compaction). To promote sufficient resistance to deformation (stability) for a particular level of traffic, the bitumen content must be kept comparatively low so that the frictional resistance of the aggregate mass will be maintained at the level necessary to carry the load. On the other hand, good durability characteristics in a mixture are promoted by a high bitumen content, together with dense aggregate gradation and good compaction. To promote flexibility in the mixture, a high bitumen content and a comparatively open (as opposed to dense) gradation of aggregate are of essence. Higher bitumen content and increased density are associated with longer service life of a bituminous mixture. To insure imperviousness, the conditions of high bitumen content, dense aggregate gradation, and good compaction are essential. High bitumen content, high percentage of fine material (filler), and good compaction can also increase the fracture strength of the mix. Table 2.1, which shows the influence of these variables on a comparative basis, indicates that a bitumen content that attempts to strike a balance between all of the desirable mix properties must be selected. In addition, for the majority of the mix properties, dense gradations appear desirable, and proper compaction in the field is emphasized. In the following sections, these important mix variables are discussed.

2.2 RHEOLOGICAL CHARACTERISTICS

Rheology is the science of deformation and flow of matter. The movements can be caused by forces or change of temperature.

The parameters which give a quantitative description of the kinematics (i.e., the time-position motion relations) of deformation and flow are strain, rate of strain, and force-time patterns. Because of the thermoviscoelastic nature of bituminous materials, the most important factors influencing stress-strain relationships are temperature and rate or time of loading. For instance, resilient moduli of bituminous materials should be evaluated in the laboratory at different temperatures and at different rates of loading. Under constant temperature conditions, it has been found that the response of bituminous concrete to loads can be described by the theory of viscoelasticity.

2.2.1 LABORATORY RHEOLOGICAL TESTS

The types of tests which have been used by investigators to measure the time- and temperature-dependent responses of bituminous mixtures to loads have included the following:[3-5]

a. Creep.

b. Stress relaxation.

c. Constant rate of strain.

d. Dynamic.

   (1) Sinusoidal variation of stress or strain with time.

   (2) Step function pulse loading where the duration of pulse (usually load) corresponds to the velocity of a vehicle (termed "repeated loading").

e. Repeated load testing.

These types of loading, together with measured responses, are shown schematically in Figure 2.1. All these tests can be used on cohesive soils. For unbound granular materials and stabilized soils (stabilized with cement or lime), repeated load tests can be used to simulate actual moving traffic loads. In the following paragraphs, these tests are described in detail.

2.2.1.1 Creep Tests. Creep tests have been performed on bituminous mixtures in compression, tension, and flexure. A constant stress is applied and strain is measured as a function of time (Figure 2.1a). Normally this test is conducted at a constant temperature. From the

creep test results, a measure of stiffness, termed "compliance," can be determined.

$$D(t) = \frac{\varepsilon(t)}{\sigma_o} \tag{2.1}$$

in which

$D(t)$ = compliance, in.^{2}/lb or cm^2/kg

$\varepsilon(t)$ = measured strain as a function of time

σ_o = constant stress

A material exhibiting linear response will have a unique relationship between compliance and time which is independent of the magnitude of the applied stress.

Creep tests have been used extensively, primarily because of their simplicity, to determine time-dependent properties of bituminous mixtures. Kenis[6] of the Federal Highway Administration (FHWA) incorporated the creep compliance in the computer program VESYS II from which pavement response could be computed. Shell[7-9] has used creep tests on bituminous materials to estimate the amount of rutting that would occur in the bituminous layers of the pavement. (Details of the referenced papers will be presented in Section 2.4 of this chapter, "Permanent Deformation Characteristics.")

2.2.1.2 Stress Relaxation Tests. In stress relaxation tests, a constant strain is applied and the resulting stress is measured as a function of time (Figure 2.1b). From these results, the relaxation modulus $E(t)$ can be determined from

$$E(t) = \frac{\sigma(t)}{\varepsilon_o} \tag{2.2}$$

in which

$E(t)$ = relaxation modulus, psi or kg/cm^2

$\sigma(t)$ = measured stress as a function of time

ε_o = constant strain

A material exhibiting linear behavior will have a unique relationship of relaxation modulus versus time which is independent of the strain

level imposed on the specimen. Also, for a linear viscoelastic material, the relaxation modulus can be obtained using numerical procedures from the creep compliance.

For a viscoelastic substance such as bituminous concrete, it is known that the stress produced by the strain will relax at a finite rate (not instantaneously). In other words, when a strain is suddenly applied to a bituminous concrete specimen and held constant, the stress induced in the specimen will decrease with time.

2.2.1.3 Constant Rate of Strain. A measure of stiffness can also be determined from constant rate of strain tests (Figure 2.1c). The relaxation modulus in this instance is computed from the slope of the stress-strain curve at a particular rate of strain, or

$$E(t) = \frac{d\sigma}{d\varepsilon} \tag{2.3}$$

in which $d\sigma/d\varepsilon$ is the slope of the stress-strain curve at a particular strain rate. By performing this type of test at different rates of strain, the modulus can be obtained as a function of the time of loading. If the material exhibits simple viscoelastic behavior, the modulus determined from Equation 2.3 should be the same as that determined from Equation 2.2 as a function of time.

2.2.1.4 Dynamic Tests. The time-dependent response of a viscoelastic substance can be determined through the application of sinusoidal loading to a specimen (Figure 2.1d). If the material is viscoelastic, the deformation resulting from the load will have the same sinusoidal variation with time but will lag behind the stress by a time represented by ϕ/ω , where ϕ is the phase angle (or phase shift) between the stress and the resulting strain and ω is the frequency of loading. From the peak amplitudes of stress and strain, a complex modulus E^* can be determined from

$$|E^*| = \frac{\sigma_o}{\varepsilon_o} \tag{2.4}$$

By measuring both the absolute value of the complex modulus and the phase angle over a range in frequencies, viscoelastic response of the bituminous concrete as a function of time is determined. Through numerical procedures,[5] it is possible to relate the complex modulus as a function of frequency to the relaxation modulus as a function of time.

2.2.1.5 Repeated Load Tests. A repeated load test uses a pulse-type loading as shown in Figure 2.1e. This type of test has been utilized by Seed and his co-workers to study the resilient properties of soils. Various forms of this test can be used to measure the stiffness characteristics of bituminous concrete at different times of loading.

The aforementioned tests are generally conducted in the laboratory at a specific temperature. A given viscoelastic material's behavior has to be measured at a number of different temperatures.

2.2.2 RHEOLOGICAL PROPERTIES OF BITUMENS AND BITUMINOUS MIXTURES

In the past decade, considerable interest has developed in attempts to determine the time of loading and temperature dependence of the stress-strain characteristics of bitumens and bituminous mixtures within a theoretical framework. One of the major objectives of this type of endeavor is to provide rheological data to be used in analyses for engineering applications where a bitumen or a bituminous mixture is a major component of a structure, such as a multilayer highway or an airport pavement.

Monismith, Alexander, and Secor[3] listed some examples of pavement behavior for which answers may be provided from this type of rheological data:

a. Influence of velocity of moving vehicle loads and temperatures on pavement surface deflections or on stresses and strains within the bituminous layers.

b. Accumulation of small deformations under repeated traffic loading leading to rutting of the bituminous layers.

c. Creep under long-time loadings.

d. Temperature-induced stresses and deformations.

A number of investigations have attempted to approach these problems within the framework of viscoelastic theory, since through this approach the above-noted factors of time and temperature can be considered. The principles of viscoelasticity have been successfully used to explain the mechanical behavior of high polymers (rubbers), and much basic information has been developed in this area.

2.2.2.1 Stiffness According to Van der Poel. To describe the rheology of bitumens, Van der Poel[10,11] introduced the concept of a stiffness modulus. In an elastic body, the strain occurs instantaneously when a stress is applied, whereas the movements in bitumens are delayed and proceed at a rate which depends on temperature. The stiffness modulus of bitumen is defined as the ratio between stress and strain:

$$(S)_{t,T} = \left(\frac{\sigma}{\varepsilon}\right)_{t,T} \tag{2.5}$$

in which

S = stiffness, psi or kg/cm^2

t = time of loading

T = temperature

σ = axial stress

ε = axial strain

For very short times of loading or at low temperatures, the behavior of bituminous concrete is almost elastic in the classical sense, and the stiffness S is analogous to an elastic modulus E. At longer times of loading or higher temperatures, the stiffness is defined by the relationship between the applied stress and the resulting strain, with regard to time and temperature.

From both creep and dynamic tests, Van der Poel developed data indicating that the stiffness of a mixture is dependent on the stiffness of the bitumen and the volume concentration C_v of the aggregate* for mixtures that contain dense-graded aggregates and bitumens and are well

---

* The volume concentration is defined as: C_V = volume of compacted aggregate ÷ (volume of aggregate + volume of bitumen).

compacted (approximately 3 to 5 percent air voids). Thus, with a knowledge of the penetration and softening point (ring-and-ball method) of the bitumen as it exists in the mixture* and the volume concentration of the aggregate, an estimate of the stiffness of bituminous concrete can be determined from Figures 2.2 and 2.3.

Figure 2.2 permits determination of the stiffness of the bitumen for a particular rate of loading and temperature from the penetration ring-and-ball softening point of the recovered bitumen. With the stiffness of the bitumen known, the stiffness of the mixture can be determined from Figure 2.3 using the volume concentration of the aggregate.

More recently, Heukelom and Klomp[12] have examined Van der Poel's method in more detail and have suggested that Figure 2.4 gives a reasonable estimate of the stiffness of bituminous concrete provided the stiffness of the bitumen is determined from Figure 2.5,** a modification of Van der Poel's nomograph (Figure 2.2). Figure 2.4 is based on

$$\frac{S_{mix}}{S_{bit}} = \left\{1 + \left[\left(\frac{2.5}{n}\right)\left(\frac{C_v}{1 - C_v}\right)\right]\right\}^n \qquad (2.6)$$

in which

S_{mix} = stiffness of the bituminous mixture, kg/cm^2

S_{bit} = stiffness of the bitumen, kg/cm^2 (determined from Figure 2.5)

$n = 0.83 \log \frac{400,000}{S_{bit}}$

The terms are applicable to well-compacted mixtures with about 3 percent air voids and C_v values ranging from 0.7 to 0.9.

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* This can be ascertained by extracting and recovering the bitumen from the mixture.

** Figure 2.5 is a modification of Figure 2.2 in that the stiffness is determined directly in kilograms per square centimetre rather than in newtons per square metre; in addition, the lines for negative penetration indices in Figure 2.5 are in a different location than those in Figure 2.2.

Many researchers have presented data to compare stiffnesses of bituminous mixtures determined from laboratory tests and those estimated from the nomograph can be found in Chapter 2 of Finn.[1] It appears that this approach may be an expedient and accurate method for determining stiffness, provided the mixtures under consideration fall within the range for which the nomograph was developed (i.e., well-compacted, dense-graded, surface course mixtures).

2.2.2.2 Rheological Properties of Bitumen. According to Heukelom,[13] deformation of bitumen can be divided into three parts: elastic strain, viscous strain, and delayed elastic strain. These are explained in the following paragraphs.

Asphaltic bitumens are basically built up from hydrocarbon molecules. When a bitumen is loaded, all the hydrocarbon molecules show instantaneous deformations, the bulk effect of which can be described with a modulus of elasticity E. The value of E is practically independent of time and temperature in the range to be considered. Under an external stress σ, all bitumens show an elastic strain

$$\varepsilon_e = \frac{\sigma}{E} \tag{2.7}$$

throughout the bitumen structure.

The colloidal structure of asphaltic bitumens leads to the assumption of a liquid phase of free molecules in which molecule agglomerates are dispersed. Forces applied to a bitumen are transmitted through the liquid phase to the molecule agglomerates so that, statistically speaking, the stresses in the two phases become approximately equal to the external stress σ.

At a constant external stress, the liquid phase having a bulk viscosity η will develop a viscous strain

$$\varepsilon_v = \frac{\sigma t}{3\eta} \tag{2.8}$$

which depends on the loading time t and on the temperature, since the viscosity is a function of temperature $\eta(T)$.

The molecule agglomerates show delayed elastic strains ε_d which start in the viscous way but gradually reach elastic equilibrium. The time required to reach equilibrium is expressed as the retardation time. The retardation times of the molecule agglomerates are distributed over a wide range. Hence, the delayed elastic deformation cannot be computed a priori. If the bulk effect is expressed in a modulus of delayed elasticity $D(t,T)$ the delayed elastic strain amounts to

$$\varepsilon_d = \frac{\sigma}{D} \tag{2.9}$$

As a first approximation, the three characteristic strains can be considered to occur independent of each other. In this case, the total strain equals the sum of these three; thus,

$$\varepsilon = \varepsilon_e + \varepsilon_v + \varepsilon_d \tag{2.10}$$

If this value of ε is submitted in Equation 2.5, the stiffness modulus can be split up in accordance with the characteristic deformations, yielding

$$\frac{1}{S} = \frac{1}{E} + \frac{t}{3\eta} + \frac{1}{D} \tag{2.11}$$

This equation expresses the viscoelasticity of bitumens. At $t \approx 0$, the term with E predominates and the behavior is nearly elastic. At $t \approx \infty$, the term with η is the largest and the behavior is viscous. At moderate loading rates (the magnitude of which depends on temperature), the delayed elastic effects exert their influence upon the transition from elastic to viscous behavior. When the retardation times are distributed over a wider range, the transition range is accordingly wider. Figure 2.6 shows a simplified diagram illustrating the dependence on time of loading of stiffness of bituminous material for a particular temperature.

A number of researchers, in addition to Van der Poel, have been concerned with establishing the rheological characteristics of bitumens.

Monismith and Secor[4] gave a summary of these investigations, and they are presented in the following paragraphs.

A number of investigations have demonstrated the viscoelastic nature of bitumens for small deformations.[14-24] In general, these materials display elastic, retarded elastic, and viscous behavior under load. Saal[14,15] has suggested that the mechanical behavior of bitumens can be broken down into three general classes, all of which can be approximated by Burger's model or some modification thereof:

a. Sol-type bitumens. Burger's model with G_1 and η_1 approaching infinity.

b. Elastic-sol type bitumens. Behavior represented by Burger's model with all four elements.

c. Gel-type bitumens. Burger's model with η_1 approaching infinity since this type of material shows no permanent deformation.

While these simplified models approximate the rheologic behavior of the three classes of bitumen, Saal[14,15] has emphasized that really satisfactory representation requires that the model be composed of a number of Voight elements in series with a Maxwell element (a more general form of Burger's model).

Brown and Sparks[16,17] have indicated that four Voight elements in series with a Maxwell element satisfactorily represent the behavior of hard bitumens in creep. They further indicate that four Maxwell elements in parallel with a simple Voight element satisfactorily represent results of tests on the same bitumens in stress relaxation. Actually both models are equivalent. However, the first lends itself to the evaluation of creep behavior, and the second to stress relaxation.

Several researchers[18,19,22,24] have used vibratory tests to assess the viscoelastic response of bitumens over a range in frequencies of load application. By applying sinusoidally varying stresses or deformation, they have developed complex representations termed complex moduli or complex compliances for a variety of bitumens. In general, their results for viscoelastic materials are stated in terms of a storage modulus (based on the component of strain in phase with the stress) and a loss modulus (based on the component of strain 90 deg out of phase with the

stress). The complex modulus is equal to the vector sum of these two moduli. It has been shown that the complex modulus can be related to Burger's model. Rather than use this method and thus develop the E's and the η's for the simple model, all of the above-mentioned investigators have left the complex modulus in general terms.

These investigators have also considered the effects of temperature and have demonstrated by means of the method of reduced variables[25] that all of the relaxation mechanisms for a given bitumen (even though a generalized model is used) apparently have the same temperature dependence, a point that may be useful when considering the effect of thermal stress.

Kuhn and Rigden,[18] based on studies using a vibrating reed, have developed complex moduli for a variety of bitumens and have demonstrated the change in rheological behavior of one bitumen after 5 years in service in a pavement.

The studies cited above serve to establish that bitumens can be treated as viscoelastic materials, at least for small deformations. In addition, the available information would seem to indicate the usefulness of time-temperature superpositions with regard to these characteristics.

2.2.2.3 Rheological Properties of Bituminous Mixture. The following material was extracted from Wood and Goetz.[26]

In addition to the work of Van der Poel[10] in the use of the nomograph relating stiffness of the bitumen to that of the mixture, Nijboer[27] and Saal[28] have made use of this work in studying the behavior of bituminous concrete in pavement structures.

Mack[29,30] and Wood and Goetz[26] have presented data for sheet bitumen mixtures subjected to creep loading in compression. Mack states that in creep these materials exhibit:

a. An instantaneous elastic strain which is independent of time.

b. A retarded elastic strain which is a function of time.

c. A plastic deformation whose rate decreases with time.

Wood and Goetz obtained the same type of data for creep loading, and, in addition, they found that for unloading the mixture under investigation

exhibited instantaneous recovery, retarded recovery, and permanent deformation. By using Burger's model, they found that ε_1 and η_1 obeyed the laws of linear viscoelasticity for limited stresses and small deformations.

Secor and Monismith[31,32] have presented data from triaxial compression tests in creep, in stress relaxation, under a constant rate of strain, and under repeated load for one bituminous concrete. They found that the four-element model suggested by Kuhn and Rigden[18] was capable of providing reasonable agreement between theory and test data over the entire range of loading conditions considered. The four-element model is similar to the three-element model, except that a dashpot is added in series with the three-element model.

Pister and Monismith[33] have shown that constant rate of strain triaxial test data (for small deformations) can be closely reproduced by a generalized Maxwell model. From tests on mixtures using two different bitumens, they showed that the method of reduced variables is adequate to develop a single generalized stress-strain curve for each mixture and that the temperature dependence of the viscoelastic properties of the mixture is dependent on the bitumen.

Hubrecht,[34] using the sonic technique, has evaluated the real part (or storage modulus) of the complex modulus for a number of different bituminous mixtures. He found that the storage modulus is: (a) inversely related to void content, (b) little affected by the gradation of the fine aggregate for sheet bitumen, (c) affected by the characteristics of both the coarse and the fine aggregate for bituminous concrete, (d) affected by the rheologic characteristics of the bitumen at a given temperature, and (e) reduced with an increase in temperature, though such a reduction is also influenced by the type of bitumen. Hubrecht's studies were limited, however, to very short loading times and to temperatures less than 77° F.

As with bitumen, the data available on bituminous mixtures suggest that viscoelastic analysis may be applicable for small deformations. Moreover, the data also indicate that the temperature dependence of this behavior may be directly related to that of the bitumen.

The following material was extracted from Finn.[1]

Secor and Monismith[5,35] and Monismith et al.,[36] from creep and relaxation tests in tension, compression, and flexure, have shown that a satisfactory measure of stiffness, accurate at least to the level required by engineering applications, can be obtained by considering bituminous concrete to be a linear viscoelastic material which is thermorheologically simple. Data illustrating this behavior are presented in Figures 2.7-2.10. A material exhibiting thermorheologically simple behavior is illustrated in Figure 2.11, in which the position of the modulus curve, but not its shape, is altered on the time scale due to temperature change. For this material, a reference temperature may be chosen and a reduced time defined so that the modulus curves in Figure 2.11 for the various temperatures superimpose when plotted versus the reduced time.

Figure 2.7 shows creep compliance data for bituminous concrete in tension. These data were shifted horizontally to obtain the extended time curve of compliance versus reduced time shown in Figure 2.8. The reduced time is obtained by dividing the real time by a term called the shift factor A_T, which is defined as

$$A_T = \frac{t_T}{t_o} \tag{2.12}$$

in which

t_T = time required to observe a phenomenon (such as creep compliance in Figure 2.7) at temperature T

t_o = time required to observe the same phenomenon at the referenced temperature

In these figures, it should also be noted that at low temperatures and/or short loading times this material approaches essentially elastic behavior with a compliance of the order of 2 to 3×10^7 sq in./lb and a corresponding modulus of the order of 4×10^6 psi.

Figure 2.9 shows creep compliance data obtained from compression tests for the same mixture. These data have been converted by a numerical procedure[5] to develop the relaxation modulus and are compared with the

relaxation modulus determined directly from stress relaxation tests in Figure 2.10; the computations are based on the assumption of linear viscoelastic behavior. Although there exist some discrepancies at short loading times, the comparison appears to be reasonable.

Studies of the applicability of linear viscoelasticity and thermorheology to bituminous mixture behavior have been conducted by Papazian[37] and Pagen and Ku[38] using sinusoidal loading and creep tests. Figure 2.12 shows typical data obtained in their investigations for the relationships between frequency and both the complex modulus and the phase shift for a bituminous mixture. It should be noted that there is a large range in the modulus value, in this case E^* , as a function of frequency (or time).

The applicability of the time-temperature superposition principle (related to the thermorheologically simple behavior) to bituminous mixture behavior has also been demonstrated by Pagen and Ku.[38] Creep moduli (the inverse of the creep compliance) were determined and plotted as a function of time of loading and temperature. These curves were then shifted to produce the composite curve at a reference temperature for an extended time range.

For many of the reported data on viscoelastic response of paving mixtures, behavior is defined in terms of a single load application. Pagen and Ku[38] have demonstrated that, if the response is measured after a number of load applications (mechanical conditioning), the material behavior will tend to be more reproducible, and linear viscoelastic theory may be more appropriately used to define the behavior of the bituminous mixture. Results of 5 cycles of creep loading illustrating this point are shown in Figure 2.13. From a field performance standpoint, because highway pavement is subjected to many repetitions of loads, this mechanical conditioning should probably be a part of a testing program.

The influence of mix variables on creep behavior has also been reported by Pagen and Ku.[38] In this investigation, the effects of bitumen type (temperature susceptibility characteristics), aggregate gradation,

and aggregate type have been studied. Aggregate gradation appears to have more of an influence than aggregate type (with all mixes at the same bitumen content), particularly at longer times, and the temperature susceptibility characteristics of the bitumen may have an influence at the longer times of loading.

Krokosky, Tons, and Andrews[39] have indicated that bituminous concrete exhibits nonlinear and nonviscoelastic behavior in compression, the degree of which is dependent on the magnitude of the deformation, particularly that associated with the aggregate. In stress relaxation, the aggregate movement appears to be the least, whereas in constant rate of strain it appears to be the greatest. Thus, the deviation from linear behavior is least in stress relaxation, more so in creep, and the greatest under a constant rate of strain.

These investigators have also examined the applicability of time-temperature superposition. Depending on the type of test, it was found that the shift factor A_T varies with the temperature. Davis, Krokosky, and Tons[40] have indicated that the type of mineral filler influences the response of bituminous mixtures at different temperatures. Although the shift factors for a mix containing limestone filler appear to give the same temperature dependence as predicted by viscosity data for the bitumen, the response to temperature is influenced by the presence of asbestos.

Krokosky and Chen[41] have presented data indicating that the shift factor is not too dependent on bitumen content, at least within a range of bitumen contents and temperatures that would be considered for field application.

In general, these investigators[39-41] indicate that time-temperature superposition is valid for bituminous mixtures, at least to an engineering approximation.

Table 2.2 summarizes the various methods used to measure the rheologic characteristics of bituminous mixtures. Finn[1] concluded that the stiffness of bituminous concrete is dependent both on time of loading and on temperature and can vary by a factor of about 10^3 over the range

of times and temperatures that will be expected in its application in service; i.e., from about 4×10^6 psi to 10^3 psi.

2.2.3 STIFFNESS VALUES OF BITUMINOUS MIXTURES FOR DESIGN PURPOSES

In previous sections, test methods to determine the rheological properties of bituminous mixtures have been described. In general, the stiffness of bituminous concrete is dependent on both time of loading and temperature; however, determination of stiffness is both costly and time-consuming. For design purposes, many agencies have developed sets of curves and equations to determine the stiffnesses of certain mixes of bituminous concrete for certain temperature ranges. The work of two of the leading agencies is described below.

2.2.3.1 Shell Procedure. Based on the nomograph developed by Van der Poel[11] presented in Figure 2.2, Shell researchers[42] prepared a curve relating the elastic moduli and temperature for normal highway loadings and conditions, as shown in Figure 2.14. For airport design purpose, the modulus values will have to be adjusted according to the thickness of the asphalt concrete. Generally, the higher the modulus the thicker the asphalt concrete.

From extensive field studies, it was found that the most critical condition for bituminous concrete occurs in the spring when the subgrade normally contains excessive moisture and provides the least amount of support and the bituminous layer is relatively cold and cannot withstand large strains repeatedly. The surface temperature under this condition ranges from 50 to 60° F and the corresponding modulus of elasticity for bituminous concrete is approximately 900,000 psi. When determining the pavement requirements to prevent excessive strain in the subgrade, the stiffness in the bituminous layer should be evaluated at the highest temperature expected in the field; i.e., when it contributes the least amount of resistance to deformation. In the design procedure developed by Shell, an air temperature of 95° F was selected as being representative of the average high expected in a large portion of the world. The stiffness of the layer was found to be associated with the temperature

at one-third the depth of the layer. Since the temperature gradient in bituminous concrete varies from top to bottom, the effective stiffness increases as the layer thickness increases. Figure 2.15 gives a relationship between modulus of elasticity and layer thickness for an air temperature of 95° F. For design purposes, the moduli range from 150,000 to 200,000 psi. In the selection of these moduli for design, the bituminous layer must be a reasonably proportioned, dense-graded mixture that has been properly densified. These moduli cannot be considered valid for lean sand bituminous mixes or aggregate mixes containing cutbacks or emulsions.

<u>2.2.3.2 Asphalt Institute.</u> An extensive dynamic testing program was carried out at The Asphalt Institute[43] to determine the complex modulus $|E^*|$ of bituminous bases and surfacings. The tests were conducted in the unconfined state at sinusoidal loading frequencies of 1, 4, and 16 Hz and at temperatures of 40°, 70°, and 100° F. The moduli are presented in Figures 2.16 and 2.17. Hence, for any speed and temperature of test track operations, a dynamic modulus could be predicted from the laboratory tests.

In the development of a design manual for full-depth bituminous airfield pavements, Witczak[44] used a relationship between the dynamic modulus E and temperature t which was input into a computer program to compute the response of the pavement. The relationship has the form

$$E = \frac{K_o}{K_1 t^{d_1}} \tag{2.13}$$

where K_o , K_1 , and d_1 are regression constants and, as chosen by Witczak, are equal to 3.8×10^6, 1.0046, and 1.45, respectively. Equation 2.13 is based on a regression analysis of numerous laboratory dynamic modulus test results and is applicable to dense-graded bituminous concrete mixes. The rate of loading applicable for the equation is 2 Hz which is typical of a dual-tandem gear traveling at a taxiing speed of approximately 10 to 20 mph.

2.3 FATIGUE OF BITUMINOUS MIXES

The term fatigue implies a mode of distress in a bituminous concrete pavement resulting from repeated application of traffic-induced strains. Failure of bituminous concrete surfacing resulting from repeated bending (fatigue) was early recognized by Hveem of the California Highway Department. In 1955, Hveem[45] presented definitive data relating pavement deflection and performance. On the basis of deflection measurements, he was able to suggest maximum or limiting deflections for the satisfactory performance of bituminous pavements. In this investigation, performance was based on the amount of cracking present; therefore, it can be assumed that the limiting deflection was selected to minimize surface cracking. The limiting deflections suggested by Hveem are given in Table 2.3. Although no definite number of load repetitions or service life was associated with these deflection values, it was implied that if deflections did not exceed these values during a reasonable service life, unlimited numbers of repetitions could be applied without flexural (fatigue) cracking.

Since 1955, highway engineering research literature has been replete with reports relating pavement deflection to pavement performance. For the most part, these investigations have established that bituminous pavements are subject to a loss of serviceability resulting from the cumulative effect of tensile stresses less than the ultimate tensile strength of the surfacing. Effort has been made to define the properties of bituminous surfacing which influence fatigue life, and some effort has been made in the laboratory to measure the effect of repetitive loadings on bituminous concrete specimens.

2.3.1 DEFINITIONS

Fatigue is the "phenomenon of fracture under repeated or fluctuating stress having a maximum value less than the tensile strength of the material."

Fatigue failure is often loosely considered to be the point at which the material or specimen is unable to continue to perform in a satisfactory manner. The failure or end point of a fatigue test has

been defined by investigators in many ways. For example, it may be taken as the point corresponding to complete fracture of the test specimen; the point at which a crack is first observed or detected; or the point at which the stiffness or some other property of the specimen has been reduced by a specific amount from its initial value.

Service life N_s is the cumulative number of load repetitions necessary to cause failure in the test specimen. In general, the service life as defined here has often been called the fatigue life, but it should be noted that it is a function of the manner in which failure is defined.

Fracture life N_f is the accumulated number of load repetitions necessary to completely fracture a specimen. When the failure point is complete fracture, then the service and fracture lives are identical.

Simple loading occurs when specimens are subjected to a series of unilevel stress or strain tests, and the corresponding repetitions to failure are obtained. The level of stress or strain is usually selected to be representative of that to be encountered or anticipated in a pavement structure.

Compound loading refers to a fatigue test performed with several levels of load. Deacon[46] discusses three compound loading test procedures:

a. Sequence loading. Applying different loads (usually, but not necessarily, two loads) in increasing or decreasing sequence; e.g., decreasing sequence would always start with the highest stress followed in order by the lower stresses.

b. Repeated block loading. A defined sequence of block loadings applied repeatedly until failure occurs.

c. Pseudorandom loading. A stipulated set of individual loadings randomly applied according to a predetermined probability for each load.

2.3.2 TYPE OF FATIGUE TEST

In the laboratory, fatigue behavior of materials such as bituminous concrete has been determined in a number of ways. Two of the most common are controlled load or stress and controlled deflection or strain. For tests of the controlled load or stress type, the repeatedly applied load

(stress) is maintained at a fixed maximum during each test. Relationships between stress and repetitions to failure and between strain and repetitions to failure as determined by controlled stress tests are shown in Figure 2.18a. It should be noted that strain (deformation) increases in this type of test until failure occurs; i.e., until the service or fracture life is reached.

For tests of the controlled strain (deformation) type, the repeated strain induced is maintained at a fixed maximum until the service or fracture life is reached. Inasmuch as damage is usually progressive in some continuous manner, the stress (load) will decrease with increasing load repetitions because the stiffness of the specimen will be decreased. Figure 2.18c shows the idealized behavior in this type of test.

In the controlled stress test the deformation increases from its initial value, whereas in the controlled strain test the stress is decreasing. Thus, the product of stress and strain increases in the controlled stress test and decreases in the controlled strain test. Because this product represents work, it can be seen that work (or energy) is expended more rapidly in the controlled stress test, and a shorter life results.

It will be seen that, in many cases, the service life of the specimens greatly depends on which of these two modes of testing is used.

While no efforts have been made in the past to identify and describe modes of loading between these two extremes, a relation such as that described in Equation 2.14 was suggested by Monismith and Deacon[47]

$$MF = \frac{|A| - |B|}{|A| + |B|} \tag{2.14}$$

in which

MF = mode factor

A = percent change in stress due to a stiffness decrease of C

B = percent change in strain due to a stiffness decrease of C

C = arbitrary but fixed percent reduction in mixture stiffness

The mode factor of Equation 2.14 assumes a value of -1 for controlled stress testing and +1 for controlled strain testing. For intermediate modes, it lies between the limits of -1 and +1, which is shown in Figure 2.18b.

Note particularly in Figure 2.18 that the initial stress levels and initial strain levels are identical for all three modes of loading. On the other hand, the observed service lives are considerably different and become progressively larger as the mode factor increases from -1 to +1. It is readily apparent that the most severe mode of loading is that of controlled stress since, for all but the initial load application, both the imposed stress and strain levels exceed those for other modes.

Further comparison of the effects of mode of loading for simple loading tests can be achieved by extending the range of initial stresses and strains. Hypothetical fatigue diagrams for the three modes of loading are shown in Figure 2.19. This figure indicates that over a wide range of initial stresses the mean service life increases as the mode goes from controlled stress to controlled strain.

The applicability of types of fatigue tests to actual road conditions has been considered by analyzing various types of pavement construction using layered elastic theory to investigate the effect of variations in bitumen stiffness on the stresses and strains occurring in the bituminous layer. A typical example is shown in Figure 2.20. The tensile stresses and strains were computed for many full-depth bituminous concrete pavements of various thicknesses. For each thickness, computations were made for three stiffnesses of the bituminous layer. The percent changes in stress and strain due to a stiffness decrease of a given percentage were computed, and the mode factors were determined from Equation 2.14. Figure 2.20 indicates that, as the thickness and stiffness of the bituminous layer increase, the mode factor decreases and a controlled stress condition is approached. It is therefore suggested that for thicker bituminous layers, say 6 in. or more, controlled stress testing conditions are appropriate, while for thin bituminous layers of 2 in. or less

controlled strain tests are suitable since under these conditions the strain is little affected by the mixture stiffness.

For the intermediate thicknesses, some form of testing between these two extreme modes would be appropriate. But from an engineering design approach, controlled stress tests would seem sensible since it will be seen that these give a conservative estimate of fatigue life.

The above conclusions may be explained in another manner as follows. In a controlled strain test, the strain is held at a constant value at all temperatures, and consequently B in Equation 2.14 is zero. In a controlled stress test, the stress is held at a constant value at all temperatures. Therefore, A in Equation 2.14 becomes zero. The mode factor in Equation 2.14 thus has values of +1 for controlled strain testing and -1 for controlled stress testing.

Computations were performed for six pavements to compute the radial tensile stress and strain at the bottom of the surface bituminous concrete layer. The layered elastic program was used in the computation. The load used in the computation was a 30-kip single-wheel load, and the modulus of the subgrade was assumed to be 10,000 psi. Two different thicknesses of the bituminous concrete layer were used in the computations. The computed values are tabulated in Table 2.4. It can be seen that for pavements with a thin bituminous concrete layer the strain decreases moderately because of the increase of the modulus of the bituminous concrete layer, while the stress increases moderately. In other words, controlled strain tests seem to better simulate the conditions for pavements with thin bituminous concrete layers, and controlled stress tests can better represent the conditions for pavements with thick bituminous concrete layers.

2.3.3 FACTORS INFLUENCING FATIGUE PROPERTIES OF BITUMEN AND BITUMINOUS CONCRETE

There is always considerable scatter of results in any fatigue testing of nominally identical specimens, and this is particularly so in the case of bituminous mixtures due to the inherent inhomogeneity of

the material and the unavoidable variation in specimen preparation. This means that fatigue life must be considered in a statistical manner. It is therefore necessary to test several specimens at each stress or strain level, and the results are usually plotted as stress or strain versus repetitions of load to failure using log-log scales. A straight line can be plotted passing through the mean of the plotted points for each stress level, and equations representing this relation can be written as

$$N_s = K\left(\frac{1}{\sigma}\right)^n \tag{2.15}$$

$$N_s = K\left(\frac{1}{\varepsilon}\right)^n \tag{2.16}$$

where

N_s = mean service life obtained at particular loading conditions

K and n = coefficients which can be determined using linear regression analysis techniques

σ = amplitude of applied tensile stress

ε = amplitude of applied tensile strain

It should be noted that the exponent n defines the slope of the fatigue line, and lower values of n denote a steeper line.

Pell[49] and Pell, McCarthy, and Gardner[50] have reported results of both controlled stress and controlled strain fatigue tests on bitumens. In an effort to eliminate the temperature effect, they converted stress to initial strain. This was accomplished by determining the stiffness modulus of the bitumen using the method developed by Van der Poel and computing strain as a function of the method of loading. The significant findings from this research effort were as follows:

a. A linear relationship exists between stress or strain and repetitions to failure when plotted on a log-log scale.

b. Plotting stress versus repetitions to failure, horizontal displacements in the linear relationship between stress and repetitions to failure were evident and appeared to be associated with temperature. However, plotting strain versus repetitions to failure will tend to minimize differences in test results for cold temperature (in these tests below 4° C). At the warmer temperatures, the repetitions to failure tend to increase with an increase in temperature.

c. Bitumen exhibits fatigue properties commonly associated with metals, specifically, failure under repeated load applications less than the fracture strength of the material.

d. The technique of converting from stress (in constant stress tests) to initial strain by means of a stiffness modulus will produce comparable results to tests performed in constant strain.

These researchers[49,50] also commented on the variability of the actual data. For tests conducted at 0° C, the repetitions to failure for a strain of 1.8×10^{-3} range from about 5.2×10^{4} to 8.8×10^{4}. This variability increases as the strain decreases and poses a serious problem as to the number of tests required to define the fatigue properties.

Among many other mix variables, such as aggregate type and grading, Pell[48] pointed out that the most important factors affecting the fatigue performance of bituminous mixes are stiffness, bitumen content, and voids content. These factors are discussed in the following paragraphs.

Possibly the greatest difficulty in interpreting fatigue test results arises from the fact that they are influenced by the method of testing. If specimens are tested in controlled stress, then for different stiffnesses results such as those shown in Figure 2.21a are obtained. At a particular stiffness S, the mean fatigue life can be represented by a straight line on a log-log plot of stress σ versus the number of repetitions of load N_s to cause failure. Different stiffnesses are represented by parallel lines showing that, with this type of testing, the fatigue life is highly dependent on stiffness and the stiffer the mix the longer the life.

The stiffness, defined as the ratio of stress amplitude to strain amplitude, is dependent on the temperature and rate of loading. If the results of the fatigue tests under controlled stress are replotted in terms of strain ε as shown in Figure 2.21b, it has been found for a wide range of temperature that all the results from different stiffnesses coincide, indicating that strain is a major criterion of failure and that the effects of temperature and rate of loading can be accounted for by their effect on stiffness.

If identical specimens are tested in a controlled strain machine which applies an alternating strain of constant amplitude, results such

as those shown in Figure 2.21c are obtained. Here it can be seen that, although the lines at high stiffnesses, S_1 and S_2, say, coincide, those at lower stiffnesses show the reverse effect of stiffness from that found in controlled stress tests.

The reason for this is that the mode of failure is different in the two types of tests. In the controlled stress test, the formation of a crack results in an increase in actual stress at the tip of the crack due to the stress concentration effect, and this leads to rapid propagation and complete fracture of the specimen and termination of the test. In the constant strain test, on the other hand, cracking results in a decrease in stress and hence a slow rate of propagation. At low stiffnesses, and hence low stresses, the measured fatigue life includes a considerable length of time necessary to propagate a crack or cracks sufficiently to reach an arbitrary state at which the specimen is considered to have failed (service life).

If measurements of stiffness are taken during a controlled strain test, it is found that the stiffness reduces with increasing numbers of load repetitions at low stiffnesses (i.e., high temperatures), and this, no doubt, is partially due to formation of small cracks. At high stiffnesses, coincident with lines for S_1 and S_2 in Figure 2.21, there is a negligible fall in stiffness during a fatigue test.

The slope of the fatigue line represented by Equations 2.15 and 2.16 appears to depend on the stiffness characteristics of the mix and the nature of the bitumen. Mixes having high stiffnesses and linear behavior result in flatter lines. This type of behavior is characteristic of dense surfacing course mixes having a relatively high content of a harder bitumen. The leaner base course mixes made with softer grades of bitumen show considerable nonlinearity, particularly at higher stress levels, and these mixes have a steeper fatigue line.

Although the logarithmic strain-service life relationship is usually shown as a straight line, it is probably in fact curvilinear, particularly at high strains where nonlinearity is apparent.

If the method or conditions of testing are such that considerable crack propagation takes place during the test, then the line representing the service life of specimens will be steeper because the rate of crack propagation depends on the stress level. This is likely to occur at higher temperatures (lower stiffnesses), particularly under controlled strain testing.

In general, increased stiffness results in a longer life at a given stress level in controlled stress testing and a shorter life in controlled strain testing at a given strain level.

It therefore follows that any mix variables which affect the stiffness are also going to affect the fatigue life of bituminous mixes. These variables are aggregate type and grading (including filler); bitumen type, hardness (viscosity), and content; degree of mix compaction; and resulting air void content. The two factors which appear to be of primary importance are bitumen content and voids content.

Increasing voids reduce the fatigue life markedly, as shown in Figure 2.22. The effect of voids is two-fold: increasing voids will result in reduced stiffness and increased stress concentrations. Because of this, the detrimental effect of voids is likely to be more apparent in controlled stress testing or controlled strain testing at low temperatures. If increasing the bitumen content reduces the voids, then the fatigue life will be increased. But if the mix already has negligible voids, then more bitumen will reduce the stiffness, resulting in increased strain and hence a reduced life under controlled stress testing (see Figure 2.23).

The general effect on the strain-service life relationship of altering the bitumen and filler content of a particular mix is illustrated schematically in Figure 2.24. For a lean mix, increasing the bitumen and filler content will result in a stiffer material and hence smaller strains and longer life. However, if too much bitumen is added, the stiffness will be reduced and hence an optimum fatigue life will be obtained.

In conclusion, it may be stated that, for good fatigue performance for thick bituminous construction, a mix of maximum stiffness should be

the objective and the quantities of filler and bitumen should be such that a condition of maximum tensile stiffness associated with minimum voids is produced.

The fatigue characteristics discussed above have been obtained from tests carried out under simple loading conditions which mainly apply continuous repetitions of loading of particular magnitudes. More recent work reported by Raithby and Sterling[51,52] and by Van Dijk et al.[53] shows considerable beneficial effects of strain recovery if periods of rest are injected between each load pulse. These findings mean that laboratory tests using continuous repeated load pulses may well underestimate the fatigue life to cause initiation of cracks in practice.

2.3.4 CUMULATIVE DAMAGE

In recent years, the cumulative damage theory based on Miner's hypothesis of a linear summation of cycle ratios has been used by many researchers[54-58] to evaluate the effects of repeated load applications on the fatigue properties of pavement materials. Design methods based on empirical correlations between subgrade strength and a given design wheel load are considered to be inadequate because they do not consider the important factor of traffic repetitions which contributes so significantly to pavement failure.

According to Miner's hypothesis, if the traffic forecast yields an estimation of n_{ij}, i.e., the predicted number of applications (or coverages) of aircraft load i on the pavement in a particular physical state (for a particular month or pavement temperature range during the year) j during the design life, if this load is repetitively applied to the pavement in this state until the pavement fails, and if N_{ij} represents the number of applications before failure, the total cumulative damage D predicted during the design life is

$$D = \sum_{j}\sum_{i} \frac{n_{ij}}{N_{ij}} \tag{2.17}$$

If d_{ij} is the damage induced in the pavement by 1 application of the i^{th} load while the pavement is in the j^{th} physical state, Equation 2.17 can also be written as

$$D = \sum_{j}\sum_{i} d_{ij}\, n_{ij} \qquad (2.18)$$

Failure occurs if D equals or exceeds one.

Since both the magnitude of aircraft loads and the physical state of the pavement continuously vary for in-service pavements, N_{ij} cannot be measured directly and has to be estimated from an established failure criterion. Such a failure criterion usually relates the level of applied strain ε_{ij} to performance (the number of load repetitions to failure). Therefore, the purpose of a cumulative damage analysis is simply to evaluate the amount of fatigue damage expected to accumulate in a trial pavement during its design life.

Deacon used the cumulative damage theory to determine load equivalencies in flexible pavements[59] and equivalent passages of aircraft with respect to fatigue distress of airfield pavements.[60] In the newly revised design manual for full-depth asphalt pavements for airfields of The Asphalt Institute,[61] Witczak applies the cumulative damage theory to evaluate the effects of both differing aircraft types and variable traffic levels associated with each aircraft for any given anticipated air carrier traffic mixture.

In the application of Miner's hypothesis, no allowance is made for intervals of "no-load" or rest periods. It may be assumed that the effect of such intervals will be beneficial or at worst of no consequence. If the effect is beneficial, then the rule, as it stands, is conservative for design purposes.

Since Miner's hypothesis is a linear summation of cumulative damage, the order of application of aircraft loads of various magnitudes has no effect on the computed value; i.e., for a given traffic forecast, which includes both heavy and light aircraft loads, the pavement will receive the same amount of total damage at the end of its design life whether the heavier aircraft loads or the lighter aircraft loads are applied during the first part of its design life. However, it is generally recognized that lighter traffic applied to the pavement during the early part of its designed life can be considered to be beneficial to the pavement, instead of causing damage.

2.3.5 DESIGN APPLICATION

To use the stiffness modulus as an indicator of fatigue properties, it is first necessary to establish a correlation between these two factors. Unfortunately, this correlation depends on the mode of testing (stress or strain), as indicated previously. Finn[1] summarizes the effects of those factors influencing the fatigue properties of bituminous mixes, according to the type of test performed. These factors are shown in Table 2.5.

Examination of laboratory data indicates that there is an absence of information as regards interaction of those factors which affect stiffness. For example, if aggregate gradation were adjusted to result in a denser mix, an increase in the stiffness would be expected. If the bitumen penetration were simultaneously increased, a decrease in the stiffness would be expected. The net effect on the fatigue life, however, is not always known. Stated in a different way, two specimens of bituminous concrete mixes with the same stiffness could have different fatigue properties due to variations in aggregate gradation, bitumen grade, void content, etc. On this basis, the only reliable way to determine the fatigue life is to perform fatigue tests.

To minimize fatigue failure in the bituminous concrete layer caused by the repetitive application of traffic loads, many agencies have adopted the criterion of limiting the magnitude of radial tensile strain at the bottom of the bituminous concrete layer. Based on layered elastic analysis, Shell[62] limited the tensile strain value to 230 μin./in. computed with the modulus of the bituminous concrete E_1 at a value of 900,000 psi.

Many agencies have developed failure criteria for bituminous concrete based on laboratory fatigue tests. The criteria shown in Figures 2.25, 2.26, and 2.27 were developed by Dorman and Metcalf,[63] Witczak,[64] and Monismith and McLean,[65] respectively. If the design temperature of the bituminous concrete (or the design modulus) and the tensile stress or strain in the bituminous layer (computed using layered

elastic analysis) are known, the failure stress repetition level can be estimated. Based on the estimated relationship between laboratory and field conditions, the failure stress repetitions of a particular pavement can then be estimated.

Deacon[67] commented on laboratory testing in the development of failure criteria from laboratory fatigue testing of bituminous mixtures. Although the comments are for bituminous mixtures, they are also applicable to other pavement materials.

a. One of the major difficulties is that of defining failure in the laboratory in such a way as to be compatible with failure as defined for the in-service pavement. Brown and Pell[54] suggest that in-service pavement life (repetitions to failure for a given strain level) is of the order of 20 times the life of a test specimen in the laboratory. Thus, perhaps the best that can currently be achieved with laboratory derived failure criteria is an estimate of crack initiation in the in-service pavement. Few techniques are available for quantitatively estimating the progression of cracking in a pavement or for considering various levels of terminal serviceability.

b. Laboratory fatigue specimens are conventionally subjected to either of two types of repetitive loading, controlled stress or controlled strain, depending upon whether stress (load) or strain (deflection) is controlled during testing. Unfortunately, the number of load repetitions to failure is extremely dependent on the type of test. It has been hypothesized that in-service pavements are subjected to a type of loading intermediate between these two types and that controlled stress and controlled strain loadings merely represent end points of an infinite spectrum of possible modes of loading. In the absence of suitable means for defining and applying intermediate modes, the problem of which form of laboratory testing to use in design procedures will continue to exist.

c. Laboratory testing requires selection of a frequency of loading (which is greater than that normally encountered by a pavement in service) and, for pulsating loads, a load duration. These as well as other laboratory loading variables significantly affect the number of repetitions to failure. The possibility that rest periods can beneficially alter fatigue response is another variable complicating use of laboratory derived failure criteria.

d. There are certain simplifications in multilayered elastic analyses which can cause a departure of predicted from

actual pavement response. One of these is that a theoretical analysis normally assumes the pavement to have unlimited lateral dimensions and allows no lateral variation in material properties. Thus, for example, no means are available for readily treating pavement edge support or differential subgrade moisture conditions. Failure criteria derived from in-service pavements would seem intuitively to account for these and other such discrepancies between theory and practicality.

e. Most analyses of highway pavements assume perfect tracking of vehicles; i.e., they do not treat the transverse of lateral distribution of vehicle placement. This simplifying assumption may lead to erroneous results if laboratory fatigue criteria are used.

Recently, Witczak[68] completed an extensive study in comparisons of various layered theory design approaches to observed full-depth bituminous airfield pavement performance. The observed performance was at Baltimore-Friendship International Airport. He concluded that the problem of pavement analysis is confounded by the recognized fact that different test methods and procedures may be used to define the same response. For example, several different methods are available to measure the modulus of a bituminous concrete mix at a given temperature. Although most of these methods yield results within the same general magnitude, differences in computed stresses and strains at a given temperature for the various bituminous concrete modulus relationships defined may be quite significant. Also, different analytical procedures may be employed to determine or compute the predicted repetitions to failure.

Figure 2.28 summarizes the three different moduli-temperature relationships defined for bituminous base material of a selected runway. $|E^*|$ is the complex modulus, which is a linear elastic response because of its stress-independent nature; $E_{s(\sigma)}$ is the flexural stiffness modulus calculated from controlled stress fatigue tests, which is a nonlinear elastic response because of its stress-dependent nature; and $\overline{E}_s$ is the average flexural stiffness computed as the average measured stiffness for the flexural stress levels used in the fatigue tests, which is the so-called pseudolinear elastic response.

As can be seen in Figure 2.28, a rather wide range in modulus

value between the methods is indicated at any given pavement temperature. This difference is further magnified by using these data along with the yearly predicted pavement temperature distribution to illustrate the yearly frequency distribution of E_1, defined by the various modulus or temperature relationships developed. Such a plot is shown in Figure 2.29 and demonstrates, rather markedly, the wide differences in yearly percentage of time the pavement would have a modulus higher or lower than a given value, depending upon which test method was used. For example, it can be seen that for a modulus of 400,000 psi, the use of the $E_{s(\sigma)}$ relationship shows that for about 65 percent of the year the pavement modulus would be less than this value. This is in contrast to the 21 percent of the year that would result if the relationship were based upon the measured dynamic (complex) modulus test. In addition to the expected frequency distribution of pavement modulus, the effect of the computed stresses and strains, determined for a given modulus or temperature relationship, is very significantly different depending upon the test results selected. Figure 2.30 shows the differences in computed strains ε_t and ε_v as a function of pavement temperature for the various moduli relationships.

Because the use of layered theory in design must initiate with the pavement temperature as the major variable affecting the pavement modulus, it can be seen that the selection of the modulus or temperature relationship is of paramount importance in analyzing pavement performance. This appears to be especially true in any analytical approaches using cumulative damage theory.

2.4 PERMANENT DEFORMATION CHARACTERISTICS

2.4.1 LITERATURE REVIEW

In recent years, much research effort has been spent in the study of permanent deformation (rutting) in flexible pavements. Rutting can result in the loss of pavement serviceability if cracking follows the formation of ruts and rapid deterioration of the pavement follows due to the accumulation of water on the pavement surface. Under normal pavement conditions, deformations within the bituminous materials occur more

likely during the late spring, summer, and early fall because of high temperature conditions. Under winter conditions, little deformation occurs in either the bituminous material or the subgrade, due mainly to the very stiff condition of the former. During some periods, the subgrade soil may be frozen in the winter and provide firm support for the overlying bituminous material and thus reduce pavement deformation.

Hofstra and Klomp[69] investigated permanent deformation of bituminous concrete using a laboratory test track. The road structure was simplified by utilizing all-bituminous concrete construction, with 5-, 10-, 14.2-, and 20-cm (1.97-, 3.9-, 5.6-, and 7.9-in.) layers of various bituminous mixes laid directly on an 18-CBR subgrade. The mixes used were of high bitumen content to induce greater rutting than would occur in practice, and the use of a strong subgrade helped to inhibit deformation in that material.

Experiments to investigate the effect of temperature indicated that for a 5-cm (1.97-in.) layer of bituminous concrete, rutting was partly due to deformation of the subgrade but for the 10- and 14.2-cm (3.9- and 5.6-in.) layers was entirely due to deformation in the bituminous concrete. It was found that deformation of the mix was due to plastic flow of the material and was not caused by densification.

A series of tests was carried out to investigate such mix variables as bitumen type, bitumen content, and aggregate type. It was found that stiffer bitumens produced mixes less susceptible to permanent deformation, and the same effect was noted for mixes with low bitumen contents or coarse aggregates. It was also found that rut depth per wheel pass decreased with increasing numbers of wheel passes. Hence, it was concluded that the mix builds up a resistance to flow during the process of deforming under repeated loading. The authors stated that this is probably due to the bitumen being expelled from between aggregate particles producing greater interlocking, explaining why angular aggregate produces more acceptable mixes than rounded aggregate.

McLean[70] describes a methodology to permit estimation of permanent deformation in pavement structures from laboratory triaxial repeated

load and creep tests. Discussion is concentrated primarily on techniques to estimate the distortion characteristics of bituminous concrete and the use of these data together with both linear elastic and linear viscoelastic theory to predict rutting in bituminous layers of pavement structures.

Morris[71] developed a mathematical model from the laboratory experimental results to predict the rut depth of the all-bituminous concrete sections at the Brampton Test Road in Canada. The computed results match very well with the measurements. However, Morris found that the majority of the deformations occurred in the lower portion of the bituminous layer where tensile stresses exist. The conclusions of Morris's study were different from those of Hofstra and Klomp[69] and McLean.[70] The details of these works will be explained in later sections.

2.4.2 METHODS TO PREVENT PERMANENT DEFORMATION

In existing pavement design methods, there are two approaches available to prevent the distress mode caused by permanent deformation. In one method, the vertical compressive strain in the subgrade surface is limited to some tolerable amount associated with a specific number of load repetitions so as to limit the plastic deformation of the overall pavement. The Shell design method[62] falls into this category. To ensure that this strain is limited, the characterization of the material in the pavement section should be controlled through materials design and proper construction procedures (density and compaction control), and materials of adequate stiffness and sufficient thickness should be used. The other procedure involves determining a minimum layer thickness with minimum component strength and stability, thus precluding excessive shear deformation in the material. The Hveem stabilometer, Marshall test, and CBR test are used in such methods.

The following discussion is extracted from Monismith[66] and deals with available methods to prevent excessive deformation in the bituminous layers of a pavement.

2.4.2.1 Standing and Uniformly Moving Traffic. To minimize rutting under uniformly moving traffic, two of the methods in widespread use[72,73] have the capability to produce reasonably performing mixtures so long as the actual service conditions correspond to those for which the basic criteria were developed. For conditions beyond the realm of current procedures, the triaxial compression test has the potential to provide parameters which, when used with analyses of systems representative of pavement structures, can provide useful design guides. A number of investigators, as will be seen subsequently, make use of bearing capacity relationships for materials whose strength characteristics can be represented by an equation of the form

$$\tau = c + \sigma \tan \phi \qquad (2.19)$$

where

τ = shear strength
c = cohesion
σ = normal stress
ϕ = angle of internal friction

By performing triaxial compression tests at temperatures and rates of loading associated with specific field conditions, the parameters c and ϕ can be ascertained for design estimates. The analysis of Nijboer[74] can be helpful to properly define the parameters c and ϕ for design purposes:

$$\eta_{mass} \cdot \frac{d\varepsilon_1}{dt} = \frac{2 \cos \phi}{3 \sin \phi} \cdot \left(\frac{\sigma_1 - \sigma_3}{2 \cos \phi} - \frac{\sigma_1 + \sigma_3}{2} \tan \phi - \tau_e \right) \qquad (2.20)$$

where

η_{mass} = viscosity of mass
$d\varepsilon_1/dt$ = rate of application of axial strain
σ_1, σ_3 = major and minor principal stresses, respectively
τ_e = initial cohesion when $d\varepsilon_1/dt = 0$

and

$$c = \tau_e + \eta_{mass} \cdot \frac{d\varepsilon_1}{dt} \qquad (2.21)$$

For standing loads, the value of c corresponds to τ_e .

Equations 2.20 and 2.21 are used to solve for ϕ and c , respectively. Data indicate that ϕ is relatively unaffected by rate of loading, and both Nijboer[74] and Smith[75] have recommended a minimum desirable value of 25 deg. To develop ϕ values equal to or greater than this, the aggregate should be rough-textured, angular, and well-graded.

The investigations of Nijboer can be of assistance in providing mixtures with specific values of c necessary to satisfy particular loading conditions. He has shown that c increases with an increase in bitumen viscosity; is dependent on the fineness of mineral filler (minus 0.074-mm fraction); increases with an increase in the amount of filler; increases up to a point with an increase in the amount of bitumen; increases with an increase in the rate of loading; increases with an increase in mix density; and is dependent on the proportion of coarse aggregate (>1.0 mm) in the mix. More specifically, Nijboer has shown that

$$c \simeq \frac{V}{0.9} \left(\frac{FB}{0.5}\right)^{4.2} \left(\frac{D}{20}\right)^{-0.36} \qquad (2.22)$$

where

V = void factor; i.e., $[1 - (\text{air void content})^{2/3}]$ when the air void content = 0.03 and $V = 0.9$

FB = filler-bitumen factor; i.e., $\left(\frac{\text{volume filler}}{\text{volume filler + volume bitumen}}\right)$

D = equivalent particle size of filler (0.001 mm)

The triaxial compression tests appear quite useful since they provide friction ϕ and cohesion c factors which, as suggested by Nijboer,[74] can be used in a solution of the Prandtl equation for a continuous strip loading

$$q_{ult} = c \cdot f(\phi) \qquad (2.23)$$

where

q_{ult} = bearing capacity, psi or kg per sq cm

$f(\phi)$ = function dependent on ϕ ; e.g., for $\phi = 25°$, $f(\phi) = 20.7$

When q_{ult} is made equal to a specific contact pressure, c and ϕ are related as shown in Figure 2.31. In this figure, a mixture with a value of c and ϕ lying on or to the right of the curve would be adequate for vehicles equipped with 100-psi tires.[76]

Saal[28] has suggested a modification of this relationship recognizing that the bearing capacity for a circular area is larger than that for a continuous strip. The corresponding values for c and ϕ according to this relationship are also shown in Figure 2.31, which is recommended with c and ϕ derived from triaxial compression tests at slow rates of loading and high temperatures.

Smith[75] has presented a relationship between c and ϕ and bearing capacity for a circular area based on a yield criterion rather than plastic flow condition as in the above formulations. For the same contact pressure, larger values of c and ϕ are required than in the previous case, as seen in Figure 2.31. Smith also suggests a minimum angle of friction of 24 deg to minimize the development of instability from repeated loading.

The relationships suggested by Saal would appear reasonable for standing load conditions with c and ϕ determined from triaxial compression tests at a very slow rate of loading and a temperature corresponding to an average high value expected in service. For moving traffic, Smith's relationship would appear most suitable; in this case, however, the values for c and ϕ should be developed under conditions representative of moving traffic and an average high temperature expected in service.

2.4.2.2 Decelerating or Accelerating Loads. Results of one study conducted by McLeod[76] for a load with a contact pressure of 100 psi are presented in Figure 2.32. The terms P and Q are measures of

friction between tire and pavement and pavement and base, respectively. Curves A and B in this figure indicate the importance of pavement thickness in minimizing the form of instability when a frictionless contact between bituminous concrete surfacing and base is assumed $(P - Q = 1)$. As the bituminous concrete thickness increases, the ratio ℓ/t (ratio of length of tire tread to bituminous concrete thickness) decreases, resulting in lower values of c at a given ϕ to prevent instability.

When $P - Q = 0$ (full friction between pavement and base--a more practical situation in well-designed and constructed pavements) and the thickness of the bituminous concrete is in the range of 4 to 6 in. (Curve C), the more critical conditions are defined by the curve suggested by Smith[75] as shown in Figure 2.32.

Nijboer[77] and Saal[28] have considered shoving by decelerating traffic to be the accumulation of permanent parts of successive viscoelastic deformations, and these permanent deformations do occur above a shear strain of 1 percent for time and temperature conditions critical for shoving (0.33 sec and 122° F for their experience).

Using the relationship

$$S_{min} = 3\tau\frac{1}{\gamma} \tag{2.24}$$

where

τ = shear (braking) stress at surface

γ = shear strain (i.e., 1 percent)

and considering a coefficient of friction between tire and pavement of about 0.5, a minimum stiffness at this time and temperature of about 15,000 psi is indicated for a contact pressure of 100 psi.

Recent work by Valkering[78] into the effects of multiple-wheel systems and horizontal surface loads on pavement structures may provide a better framework for design against shoving. Here, attention is drawn to the fact that at high temperatures in pavements with thin bituminous layers, the shear stresses at the bituminous layer/base interface will be the highest, and that adhesion between the layers is very important if serviceability is to be retained.

For gap-graded mixes with a stone content in the range of 30 to 50 percent, Marais[79] has suggested limiting values of various mix properties to prevent permanent deformation.

Developments by Shell for the solution of stresses and deformations in elastic systems due to horizontal forces applied to the pavement surface (BISAR[78]) may provide the framework for a procedure to examine the influence of braking or accelerating stresses on distortion using a procedure similar to that suggested by Heukelom and Klomp[80] for vertical loading.

2.4.3 METHOD TO PREDICT PERMANENT DEFORMATION

The methods presented in the previous section are limited in that they do not give an indication of the actual amount of rutting which may occur under repetitive traffic loading. Unfortunately, no method presently exists whereby such estimates can be made. Promising procedures include the use of linear viscoelastic theory[81-83] and the use of linear elastic theory suggested by Heukelom and Klomp,[80] Barksdale,[84] and Romain.[85] In the layered elastic procedure, the stresses and strains are computed in the pavement structure and from these values permanent deformations in each layer of material are predicted from constitutive relationships determined by laboratory repeated load triaxial tests.

Elastic theory, together with creep data from simple laboratory tests, may also be used to estimate permanent deformation. This approach has been pursued by Shell investigators[7-9] to estimate the rutting occurring in bituminous layers.

In the Heukelom and Klomp procedure, the vertical strain distribution along a vertical axis is estimated within the bituminous layers utilizing layered elastic theory. Permanent deformation can then be determined by means of the equation

$$\delta_p = \int_0^h f(\varepsilon_v)\, dz \qquad (2.25)$$

where

δ_p = permanent deformation

$f(\varepsilon_v)$ = function relating the permanent strain ε_p to total strain ε_v ; i.e., $\varepsilon_p = f(\varepsilon_v)$

Such a technique appears useful at this time to assess, at least, the effects of changes in tire pressure and/or gear configuration (and load) on bituminous layers. In addition, it may be possible to establish limiting values for ε_p by comparing computed strains for particular field sections for which well-documented field measurements are available. Like fatigue characteristics, however, it is highly probable that any such criteria established for permanent deformation will be dependent on mixture stiffness (and thus on temperature).

In the following review, investigations at the University of Nottingham,[86,87] University of California at Berkeley,[70] University of Waterloo,[71,88] and at the Esso Laboratories in France[89] are discussed.

2.4.3.1 University of Nottingham. Repeated load triaxial tests were carried out by Snaith[86] on a dense bitumen macadam. The effect of six major variables was investigated: (a) vertical stress, (b) confining stress, (c) temperature, (d) frequency of the vertical stress pulse, (e) rest periods, and (f) bitumen content.

In confined tests, some samples developed longitudinal cracks during the test, and all unconfined samples showed a volume increase. The cracking was caused by the cyclic variation of tensile hoop strain at the surface of the sample, and would contribute to volume increase and sample failure. In confined tests, volume increase did not occur and cracking was not observed. This is more comparable with an in situ situation where restraint is offered by the large mass of material. Hofstra and Klomp[69] measured strains of up to 15 percent in situ, whereas strains measured at simple failure by Snaith were only about 2 percent, supporting the theory that adjacent material in situ prevents the cracking which hastens failure of a test sample.

The effect of confining stress was not thoroughly studied in Snaith's study. Problems arise in predicting permanent strains in the

bituminous layers if the extreme points in the layer are considered, since the range covered by Snaith's results only deals with stresses near the center of the layer. However, he suggested, as an approximation, that the layer could be considered as a whole, and stress conditions at the center taken as a mean, since Hofstra and Klomp[69] found that the permanent strains were reasonably constant with depth. Snaith found, when considering a pavement with a 200-mm layer, such as that tested by Hofstra and Klomp, divided into three sublayers, that he could determine the permanent strain in the two top sublayers and obtain good agreement with measurements made by them. The computer program BISTRO was used in the elastic analysis to calculate the stresses at the center of each sublayer, using appropriate values of stiffness and Poisson's ratio.

The following conclusions were drawn from Snaith's work on repeated loading of dense bitumen macadam:

a. An increase in temperature caused a significant increase in strain.

b. An increase in vertical stress caused an increase in strain.

c. An increase in confining stress caused a decrease in strain.

d. The level of static confining stress which gave the same strain as the dynamic confining stress was approximately equal to the mean level of that stress.

e. Realistic changes in the relative lengths of vertical and confining stress pulses did not affect the strain.

f. The rate of strain appeared to be time-dependent at frequencies above 1 Hz.

g. Rest periods between vertical stress pulses had negligible effect on strain.

h. An optimum bitumen content of 4 percent existed for maximum resistance to strain between 10° and 30° C. At 40° C, better resistance was achieved with a 3 percent bitumen content.

i. The results obtained from laboratory tests when applied to the pavement design problem produced reasonable values of rut depth.

Conclusion h indicates the relative importance of aggregate interlock and bitumen viscosity in resisting permanent strain. The former is paramount at high temperatures.

So far, when calculations of permanent deformations have been made, one combination of the principal stresses has been used at the center of each layer, whereas, in situ, this combination will change at a particular point each time a vehicle passes. It remains to be seen whether the adoption of a standard wheel load can accurately represent the wide variation of random applications of wheel loads. A limited test program is under way at Nottingham[90] using Snaith's equipment suitably modified to investigate the value of this investigation. In particular, the effects of temperature change and vertical stress change during a test are being investigated.

2.4.3.2 University of California at Berkeley. Repeated load triaxial tests were carried out on bituminous concrete specimens by McLean.[70] An attempt was made to cover the whole range of stresses to be encountered in situ by adopting three types of tests to reproduce conditions at the top, center, and bottom of bituminous layers. These were triaxial extension (cycling lateral stress only), unconfined compression (cycling vertical stress only), and triaxial tension (cycling vertical stress in tension and lateral stress in compression).

The permanent deformation, strain, and stress states of a 200-mm layer of material such as that used by Hofstra and Klomp[69] were investigated by applying the theoretical model derived from the experimental results and using Barksdale's approach.[84] Good agreement with Hofstra and Klomp's results was noted. In particular, the same form of rut depth versus load repetitions curve was obtained. Figure 2.33 shows the distribution of elastic stresses and strains and permanent strains with depth at a particular condition. The similarity between the distributions of permanent strain, stress difference, and elastic strain could be significant. Unlike the observed results of Hofstra and Klomp, the distribution of permanent strain was not uniform, possibly due to the simplifications adopted by McLean with regard to loading time.

The following conclusions were drawn from the investigation:

a. The subgrade stiffness appears to have little influence on the accumulation of permanent deformation in the bituminous layers--at least for the range of stiffness examined.

b. Stiffness exerts a significant influence on rutting in bituminous layers.

c. Like the measurements of Hofstra and Klomp, the calculation procedure indicated that rut depth in the bituminous layer was independent of layer thickness for the range examined.

2.4.3.3 University of Waterloo. Research was carried out at the University of Waterloo[71,88] in Canada for the prediction of rut depth by using a combination of linear elastic theory and the results of laboratory triaxial testing of bituminous concrete. Two series of laboratory tests were carried out on a bituminous concrete: compression tests and tension tests. Both involved the application of a cyclic confining stress and this was combined with cyclic axial compressive and tensile stresses, respectively. Both vertical and lateral deformations were measured. For the compressive tests, the vertical deformation was of interest for prediction purposes, while the lateral deformation was relevant for the tensile tests as this represented the vertical in situ deformation in the lower half of the bituminous layer.

The results showed remarkably good agreement in view of the many potential sources of error both in laboratory test techniques and in application of the results to practice. A typical result showing the variation of permanent deformation along a pavement section is shown in Figure 2.34. It can be seen that nearly all permanent deformation in a bituminous layer occurs in the lower half of the layer and results from the action of tensile lateral stresses, which is in contrast to the observations of Hofstra and Klomp[69] and the predictions of McLean.[70]

In a recent paper, Brown[91] commented that the methods used by Morris[71] and McLean[70] may not be as sound as their good respective predictions for permanent deformation at the surface imply. Brown suggested a procedure involving the use of stress invariants which are functions of the principal stresses, mean normal stress, and octahedral shear stress, but are independent of the orientation of the axes. Corresponding strain invariants can be determined from the laboratory tests and better estimates of the in situ vertical strain obtained.

Using this approach, some of the inherent disadvantages of the triaxial test can be overcome. In particular, the tension zone stresses

in a bituminous layer can be reproduced more accurately under the conditions when large permanent deformations are likely. Lower temperatures and thin layers, however, do still present a problem.

2.4.4 RELATIONSHIP BETWEEN RUTTING AND CREEP TESTING

The use of creep tests on bituminous materials together with elastic layer theory to represent the response of the pavement structure to load is an alternative approach proposed recently by Shell investigators[7-9] to estimate the amount of rutting occurring in the bituminous layers of the pavement. Three phases may be distinguished in the work carried out by Shell:

a. A study of the creep properties of bituminous mixes.[7]

b. A correlation of rutting and creep tests on bituminous mixes.[8]

c. The systematic discrepancies observed in Phase 2 were studied with regard to the main points of difference between the creep and rutting tests; i.e., unconfined-confined and static-dynamic tests.[9] A design procedure was then proposed for estimating, from the creep behavior of a mix in the laboratory, the performance of the actual pavement based on the "predetermined criteria of the pavement deformation and desired service life."

The overall aim of the work, as stated by Hills,[7] was to provide a procedure whereby rut depth could be predicted when the bituminous mix and the in-service conditions are known. To this end, creep tests were carried out in a modified version of a soil consolidation apparatus. The ends of the specimens were lubricated with powdered graphite to reduce barreling.

Tests were carried out in a controlled temperature room at either 10, 20, or 30° C on specimens that were usually 200 mm in height and 60 mm square in cross section. Specimens were cut from a slab of the mix. Some tests were carried out on cylindrical specimens of "Marshall" dimensions, the load being applied in the axial direction. Failure of test specimens was defined as the point at which the rate of strain increased, and the experimental data given were confined to those parts of

the creep curves where the strains were less than the critical "failure" values. Creep tests were carried out on a range of mix compositions and, in the case of one composition, for a series of specimens that had been compacted by various methods.

Earlier work by Shell[10,92] had shown that for short times of loading and low temperatures, the stiffness of the mix S_{mix} was a function only of the stiffness of the bitumen S_{bit} and the volume concentration of the aggregate C_v when the void content did not exceed 3 percent. The results reported by Hills[7] indicate that, at higher temperatures and longer times of loading, S_{mix} becomes insensitive to variations in the corresponding low values of S_{bit} and tends to level out to a limiting value. Furthermore, in addition to the effect of the volume concentration of aggregate, the gradation and shape of the aggregate play a role and the state and method of compaction exert a strong influence on the behavior. Other results indicate that:

a. In the case of two mixes with the same aggregate grading but with different bitumen contents and compacted in the same way, the mix with the lower bitumen content has a higher value of S_{mix} at any particular value of S_{bit}.

b. The effect of substituting crushed for rounded aggregate is to produce, at low values of S_{bit}, higher values of S_{mix}.

c. Void content of the mix cannot be used in itself for specifying the state of compaction.

Hills suggested that creep curves indicate a continuous change in the internal structure of a mix during the course of a test, and theoretical models for the deformation were developed to take account of this.

A study of the correlation between the creep and rutting properties of bituminous mixes in laboratory tests is described by Hills, Brien, and Van der Loo.[8] There were two types of rutting tests in both of which a wheel was rolled on the material in a single wheel path. In the first, rutting tests were carried out on an indoor circular test track.[69] A wheel ran at a constant speed in a circular path on a track which was 70 cm (27.6 in.) wide and on which a bituminous layer was laid on sand. The average tire contact pressure was found to be 0.5 MPa. In the other, a solid rubber-tired wheel passed back and forth over a 30- by 30- by 5-cm (11.8- by 11.8- by 2-in.) bituminous slab which lay on a rigid steel

A correction factor of 2 derived from analysis using the BISTRO computer program was used in determining S_{mix} for the rutting tests. To establish if this assumption of elastic behavior was in fact an oversimplification, parking tests were carried out with a static wheel on the test track pavement. The parking tests were carried out for 24 hours at ambient temperature and the contact stress was taken to be equal to that in the rutting test; i.e., 0.5 MPa. A comparison of the measured rutting and parking deformations at equal values of $(S_{bit})_{visc}$ indicated that the parking deformations showed the same systematic deviations from the rutting values as those calculated from the creep test. The fact that the systematic deviations in the parking test were almost a factor of 3 as opposed to a factor of 2 for the creep tests suggested that the use of an "elastic" correction was a better approximation than a procedure in which the geometry was simulated in a continuous parking or indentation test. It was thus concluded that the systematic difference between the two types of test did not result from the use of the "elastic" correction factor or from the fact that the one was confined and the other was not, but rather from the fact that the one was static and the other was dynamic.

The assessment of the "static-dynamic" contribution to the observed deviation was made by carrying out unconfined creep tests with continuous and repeated loading. The measured total and permanent deformations, or the stiffness of the mix derived from them $(S_{mix} = \sigma/e_{mix})$, were in all cases compared at equal values of S_{bit} and $(S_{bit})_{visc}$, respectively.

It was concluded that with regard to permanent deformation, the dynamic stiffness modulus of a bituminous mix is always lower than the static modulus, compared at equal values of S_{bit} and $(S_{bit})_{visc}$.

The Shell group[10,92] found that even in the most simple laboratory rutting experiment (constant speed, constant load, single wheel path, controlled temperature) it was not possible to predict rut depth with a higher accuracy than a factor of 2. The accurate prediction of rut depths on the actual road was thus considered to be extremely difficult and it was concluded that the main purpose of laboratory test methods must be

limited to the ranking of materials rather than the prediction of rut depths.

Some creep testing was also undertaken by Snaith[86] in association with his repeated load tests. The object was to see if a relatively simple test could be used to predict the permanent deformation under the more complex repeated load situation. Similar ranges of vertical stress and temperature to those used in the repeated load tests were investigated. It was intended to determine the level of static stress which gives the same creep curve as a particular dynamic stress. This has been done in Figure 2.35 where the strains after 100 and 500 sec have been plotted against the applied stresses. It was found that at low stresses the static and dynamic results are similar. However, at the higher stress levels a static stress of about 65 percent of the dynamic value would be required to produce the same strain at a particular time.

In the creep tests, the mechanism of deformation was not complicated by cracking noted for dynamic tests. Shorter lives would, therefore, be expected in the dynamic case under comparable conditions. The fact that the creep stress necessary to produce strains similar to those in a dynamic test is 65 percent of the dynamic stress rather than 50 percent supports this.

Lateral deformations were not measured in the creep tests so no measure of volume change was obtained. Hills, Brien, and Van der Loo[8] have, however, reported volume decreases in similar creep tests. Hills suggested that a different mode of failure exists in creep tests which are different from that occurring in the dynamic case where dilation takes place.

2.5 OTHER BASIC PROPERTIES

The fracture strength, durability, and thermal stress of bituminous mixtures are outlined by Finn[1] and will be given in this section.

2.5.1 FRACTURE STRENGTH

In Section 2.2 it was demonstrated that the stress-strain characteristics of bituminous concrete are both time- and temperature-dependent. From the available data, it appears that the fracture or

breaking strength of bituminous concrete also depends on these parameters.

As a factor in the design of bituminous surfacings, fracture or tensile strength appears to be of importance in three areas of design: (a) for failure under single load applications, (b) for pavement slippage wherein tensile strength would be an important consideration, and (c) for thermal stresses. Van der Poel[10] has shown that, at short times of loading and low temperatures, the breaking strength of bituminous concrete in tensile creep tests approaches 30 kg/cm^2 ($\approx$420 psi). The bitumen type also has influence on the breaking strength, particularly as influenced by temperature.

Van der Poel also presented data obtained by Lethersich[93] illustrating the influence of rate of loading and of viscosity on the breaking strength of bituminous concrete. These data approach limiting values in the same range as those obtained by Van der Poel.

Eriksson[94] has also investigated the fracture strength of bituminous concrete and has determined the tensile strength for a number of materials to be in the range 20 to 40 kg/cm^2 (280 to 560 psi), essentially the same as that reported by Van der Poel.

Rigden and Lee[95] have reported data for the tensile strength of both weathered and unweathered tars and bitumens to be in the range 25 kg/cm^2 ($\approx$350 psi) at high rates of loading in constant rate of stress tests for specimens with cross-sectional areas approaching 1 sq cm. However, they determined that the size of the specimen affects the breaking strength. Similar size effects have been observed in other materials and have been attributed to the presence of a large number of flaws in the material in larger cross sections.

Brodnyan[22] has briefly presented some tension test data on 11 representative bituminous materials used in the United States. These tests, like those of Van der Poel and Lethersich, would be considered in bulk tests because the specimens were at least 3 in. long with a 0.45- by 0.25-in. (1.12- by 0.44-cm) cross section. Brodnyan reported values of tensile stress up to about 25 kg/cm^2 ($\approx$350 psi) at 0° C for a gel-type bitumen.

The general observation that all-bituminous binders have maximum tensile strengths of the same order of magnitude in bulk has led both Van der Poel[10] and the British Road Research Laboratory[96] to the conclusion that it is possible to obtain a comparison of the susceptibility of bituminous materials to brittle fracture by measurement of the stiffness. Van der Poel suggests that brittleness effects become important when the stiffness of the material is in the range of 10,000 kg/cm^2 ($\sim$1,400,000 psi) and greater. Rigden and Lee[95] have shown that the tensile strength of tars is increased by the addition of filler. The tensile strengths determined in their tests at various temperatures and filler concentrations are given in Table 2.6.

Data obtained by Eriksson[94] show similar trends for bitumen. He found that the tensile strength of bitumen-filler mixtures increases from a value about 560 psi to approximately 1700 psi when the filler to bitumen ratio is increased from 1 to 4. Eriksson[94] also notes that the sensitivity of bitumen to stress concentrations at low temperatures is decreased with the addition of filler. The tensile strength of bituminous concrete is also both time- and temperature-dependent. Van der Poel[10] has presented data for a sheet asphalt which illustrate that at low temperatures its strength is essentially constant, on the order of 50 kg/cm^2 (700 psi), and that it decreases as the temperature is increased.

Eriksson[94] has presented data which indicate that the fracture strength is dependent upon mixture composition, specifically the filler-bitumen ratios. The tensile strength of this mixture is essentially the same as that reported by Van der Poel; i.e., approximately 50 kg/cm^2 (700 psi). Eriksson has also suggested that bitumens at low temperatures are sensitive to stress concentrations, inasmuch as the stress-strain relationship at these temperatures has no yield point. However, he notes that if filler is added the sensitivity of bitumen to stress concentrations at low temperatures decreases.

In another publication, Friksson[97] presented data for tension tests on a particular sheet asphalt for a wide range of temperatures. In general, the trend toward decreasing strength with increasing temperature

is the same as that reported earlier by Van der Poel. It is interesting to note, however, that for very low temperatures the strength is also somewhat reduced. Eriksson has conjectured that this may be due to uneven distribution of stresses at these lower temperatures. Also, it will be noted that the maximum value of tensile strength is on the order of 50 kg/cm^2 ($\approx$700 psi) for this mixture.

Tons and Krokosky[98] have also presented data showing the influence of mixture composition, rate of loading, and temperature on the tensile strength of bituminous concrete. Utilizing 1/4-in.-maximum-size aggregate, tensile strengths as high as 40 kg/cm^2 ($\approx$600 psi) were obtained with mixtures containing a combination of limestone, dust, and asbestos as the mineral filler. Temperature and rate-of-loading effects similar to those reported by Eriksson were obtained. Typical data from their investigation are shown in Figure 2.36. Although not shown, it should also be noted that, depending on the bitumen content and the type of mineral filler, maximum values for tensile strength ranged from about 20 to 40 kg/cm^2 ($\approx$300 to 600 psi).

Heukelom and Klomp have presented data (Table 2.7) covering a range of mixture compositions in which tensile strengths as high as 100 kg/cm^2 were reported. Strain at break is also given in Table 2.7 with a minimum value on the order of 1100×10^{-6} in./in.

The British Road Research Laboratory[96] has presented tensile creep data developed for tar-filler mixtures. The results show that there appears to be an optimum bitumen content for mixtures subjected to creep in tension. At low bitumen contents, comparatively small deformations result in fracture at short loading times; but as the bitumen content increases, the deformation curves appear to reach a steady creep rate or to level out for sustained periods of time. At still higher bitumen contents, fracture again occurs in comparatively short periods of time, although the strain at break is larger than that at low bitumen contents. The British Road Research Laboratory[96] suggested that properties other than mixture stability, such as fracture strength, can also be optimized through testing.

Because of these data, both Van der Poel[10] and the British Road Research Laboratory[96] suggest that it may be possible to obtain a comparison of the susceptibility of bituminous materials to brittle fracture by measurement of their stiffness.

The addition of mineral filler to bitumen appears to increase its fracture strength. As indicated by Eriksson, this may be due to the ability of the filler to reduce the sensitivity of bitumen to stress concentrations.

For mixtures of bitumen and aggregate, the fracture (tensile) strength under rapid loading and/or low-temperature conditions is in the range of 40 to 100 kg/cm^2 ($\approx$550 to 1400 psi). Strain at fracture under these conditions is probably on the order of 1000×10^{-6} to 1200×10^{-6} in./in. Moreover, the stress-strain characteristics for these conditions may be linear to failure. Eriksson has indicated that the slope of the stress-strain curves of his mixtures under these conditions approached the dynamic stiffness of the material. Thus, as with the bitumen, stiffness may be a good criterion for determining the susceptibility of material to brittle fracture.

2.5.2 DURABILITY

Durability of bituminous concrete has been defined as the long-term resistance to the effects of aging. Specifically, for bitumen and aggregate per se, durability usually refers to the rate of change of the physical properties with time. For bituminous concrete, good durability can be described as the apparent ability to provide long-term performance without abnormal amounts of cracking and raveling. A bituminous surface could conceivably be composed of bitumen and aggregate, each completely unaffected by time, but because of poor mix design or construction would not be resistant to the abrasive action of traffic. This bituminous surface would have poor durability even though the bitumen and aggregate had good durability. Or, conversely, a bituminous mix could be made with a bitumen which hardens rapidly with time, but, by means of adjustments in mix design and construction control, would give acceptable performance.

This surface would be considered to have good durability even though the bitumen has poor durability as measured by conventional tests.

Hveem[99] has divided pavement deterioration into the following three failure categories: (a) deformation caused by traffic, (b) cracking due to effects of traffic and material properties, and (c) disintegration due to traffic, material properties, and environment. Thus, material properties are listed as major contributors to pavement deterioration.

Finn[1] summarized the factors which are important to the durability of bituminous mixtures. These factors are discussed as follows:

<u>a</u>. Mixes should be designed to provide for a maximum bitumen content without instability. This has long been axiomatic in mix design. The most positive way to attain this objective is to establish a total void requirement.

<u>b</u>. Mix designs should include minimum film thickness requirements. Campen et al.[100] have suggested a minimum film thickness of 6μ (bitumen index of 1.23×10^{-3}). Based on the results of fracture strength research reported in Chapter 3, it appears that this value could be increased appreciably to about 20μ, although the need for further research is indicated. This could possibly be accomplished by adjusting aggregate grading requirements and using bitumen of higher mixing viscosity.

<u>c</u>. Mixes should be designed to have low permeability. Limiting criteria for air or water permeability are still being studied and require further evaluation. Goode and Lufsey[101] indicate this measurement may not be necessary, providing the voids are low. Until further evaluation is accomplished, it appears that use of the air permeability device can be a useful tool to adequate densification. Some useful information as to methods for measuring air permeability is given by Ellis and Schmidt[102] and Kari and Santucci.[103]

<u>d</u>. Tests of physical properties, after exposure to water, should be performed on the bituminous mixes in cases where performance history is unknown or suspect. The moisture vapor susceptibility or immersion-compression test should be suitable until further research can develop a better test or tests.

<u>e</u>. Compaction of in-place bituminous surfacing is critically important. In view of the evidence presented, a minimum compaction requirement should be specified. Many highway agencies now require a minimum relative density of 95 percent based on a specific laboratory compaction procedure. For airfield surfacing the minimum density is sometimes raised to 98 percent. Eventually, it would seem desirable to compact

mixes initially to in-place voids contents of approximately 3 to 5 percent. Indications are that this condition would improve the long-range performance of bituminous surfacing under almost every condition, provided the mix will remain stable.

f. The grade or consistency of the bitumen to be used appears to be a more controversial decision. It has been a general rule to use the bitumen of highest penetration (softest) possible compatible with stability requirements. Several factors would tend to indicate this rule may require modification, at least under certain circumstances.

(1) Using a high penetration bitumen initially does not always ensure a high penetration after mixing and 2 or 3 years of service. Some of the satisfactorily performing bitumens on the Zaca-Wigmore project had retained penetrations of only 25 percent of their original penetration after 30 and 35 months of service. Halstead indicates that even with high retained penetrations, if the ductility is low, the bitumen may not perform as expected.

(2) To obtain increased film thicknesses, the bitumen consistency may need to be relatively high.

(3) Based on limited fatigue data, bitumens of low penetrations or high viscosities may provide better fatigue properties when used in thick bituminous surfacings (greater than 4 in.). This requires field verification; however, it appears to be worthy of consideration.

(4) Resistance to the effects of water may be increased by using bitumen of lower consistency. The available information does not extend to 40-50 or 60-70 penetration bitumens and should be researched further to include these grades.

In suggesting the harder bitumens, particularly for the thick bituminous surfacing, the engineer must be mindful of the mixing and compaction requirements and therefore must balance the need to satisfy these requirements against stiffness, film thickness, etc., as were discussed.

2.5.3 THERMAL STRESSES

Bituminous mixtures, like other engineering materials, undergo volume changes with changes in temperature. If these volume changes are restricted because of constraints such as friction between the pavement and the underlying layer or because of differential temperature changes

in the material itself, it is possible that stresses will develop which may be of sufficient magnitude to cause cracking of the pavement. If the thermal stresses are not of sufficient magnitude in themselves, they may be additive to other stresses such as those resulting from vehicle loads, which in turn could lead to cracking. Thus, under certain circumstances, it may be worthwhile to make an estimate of these temperature-induced stresses resulting from restriction of volume changes to aid the engineer in the proper design of the bituminous structure.

Finn[1] summarized data on thermal properties of bitumens, aggregates, and bituminous mixtures to indicate the characteristics required to determine temperature distributions in mixtures and to determine the magnitude of thermal stresses which might be expected because of restraint of volume changes resulting from changes in temperatures.

Above a characteristic temperature, termed the glass transition temperature, bitumens display a cubical coefficient of expansion of 5 to 7×10^{-4} per $^{\circ}$C. Below this temperature, the coefficient is reduced to 2 to 4×10^{-4} per $^{\circ}$C. With respect to the bitumen, this temperature gives a measure of the transition from elastic behavior to behavior where time effects become important. This, in turn, could have significance with respect to the behavior of paving mixtures, in that bitumens with higher glass transitions may result in mixtures where this transition from time-dependent to elastic behavior occurs at higher temperatures than for mixtures prepared with bitumens with lower transition temperatures. As Monismith, Alexander, and Secor[3] and Monismith and Secor[4] have shown, it is primarily in the range where the mixture behaves elastically that high thermal stresses may develop; thus, it is possible that the glass transition temperature of the bitumen will have significance as far as the development of thermal stresses in the mix is concerned.

The data for the coefficient of thermal expansion for mixtures indicate that its magnitude is between that of the aggregate and the bitumens, whose coefficients are at least one order of magnitude different. A value of α of about 2×10^{-5} per $^{\circ}$C would appear to be

representative of bituminous concrete, with this value being higher as the bitumen content is increased. The coefficient of cubical expansion can be taken as three times this value.

The specific heat of mixtures appears to be primarily influenced by the specific heat of the aggregate, because it occurs in such large concentrations in bituminous paving mixtures. An upper value of the order of 0.22 cal/g-°C in the range 0 to 25° C appears to be reasonable for mixtures with comparatively high bitumen contents.

Barber,[104] and Monismith, Alexander, and Secor,[3] and Monismith and Secor[4] have shown that temperature distributions at the pavement surface and within thicker bituminous concrete layers can be estimated with a reasonable degree of confidence. From a knowledge of such temperatures and the rheologic behavior of bituminous mixtures over a range in times of loading and temperatures (mixture stiffness), it has also been indicated that estimates of thermal stresses can be made. Although these estimates are by no means precise, they do give an indication to the engineer as to the probable range of temperatures where he can expect difficulties. In general, it appears that thermal stresses, by themselves, will not cause cracking at higher temperatures. However, in the lower temperature range, below freezing, it is possible that thermal stresses, whether by themselves or when added to the load stresses, may result in fracture of the mix. Thus, this situation should be considered by the design engineer where warranted.

Table 2.1

Desirable Characteristics to Optimize Mixture Properties

Mix Property	Bitumen Content	Aggregate Gradation	Degree of Compaction
Stability	Low	Dense	High
Durability	High	Dense	High
Flexibility	High	Open	--
Fatigue resistance	High	Dense*	High
Skid resistance	Low	Dense or open**	High†
Imperviousness	High	Dense	High
Fracture strength	High	Dense	High

* Assuming a heavy-duty, comparatively thick layer of bituminous concrete.

** Both types of gradations have good skid resistance characteristics. What appears to be more important is the texture of the aggregate particles.

† Although compaction is not normally indicated for this property, it is implied to ensure that aggregate particles will not dislodge under the tractive forces applied to the surface.

TABLE 2.2

SUMMARY OF METHODS TO MEASURE RHEOLOGIC BEHAVIOR OF *BITUMINOUS CONCRETE* (After Finn[1])

METHOD OF TEST	INPUT	MEASURED RESPONSE	MEASURE OF RHEOLOGIC BEHAVIOR	REMARKS
Creep:	Tension and compression:	Strain as a function of time, $\epsilon(t)$	(a) Creep compliance, $D(t) = \frac{\epsilon(t)}{\sigma_0}$	(a) Necessary to use superposition principle* to relaxation modulus, $E_r(t)$, from compliance, $D(t)$.
(a) Axial loading	Constant stress, σ_0		(b) Creep modulus, $E_c(t) = \frac{\sigma_0}{\epsilon(t)}$	(b) $E_c(t) = E_r(t)$ only at short and long loading times.
(b) Bending	Flexure: Constant moment		(c) Creep modulus in flexure, $E_B(t) = \frac{1}{I} M \frac{[\epsilon_t + \epsilon_c](t)}{h}$	(c) $E_B(t) = E_r(t)$ only at short loading times for *bituminous concrete*.
Relaxation:	Tension and compression:	Stress as a function of time, $\sigma(t)$	Relaxation modulus, $E_r(t) = \frac{\sigma(t)}{\epsilon_0}$	(a) Analogous to van der Poel's stiffness.
(a) Axial loading	Constant strain, ϵ_0			(b) Difficult to perform true stress relaxation test on *bituminous concrete*
Constant rate-of-strain:	Tension and compression:	Stress and strain, σ and ϵ	Relaxation modulus, $E_r(t) = d\sigma/d\epsilon$ at different values for $d\epsilon/dt$	
(a) Axial loading	Constant rate-of-strain, $d\epsilon/dt$			
Dynamic loading	Tension and compression:	For stress input:	Complex modulus, $\lvert E^* \rvert = \sigma_0/\epsilon_0$ and phase shift Φ for a range in frequencies	(a) $\lvert E^* \rvert = E_r(t)$ only at short and long loading times. Necessary to use another form of superposition principle to determine $E_r(t)$ for intermediate times.
(a) Axial loading	Sinusoidally varying stress, $\sigma = \sigma_0 \sin \omega t$; or sinusoidally varying strain $\epsilon = \epsilon_0 \sin \omega t$. Range in frequencies	Sinusoidally varying strain, $\epsilon = \epsilon_0 \sin(\omega t - \Phi)$ at particular frequency, where Φ = phase shift ω = frequency		(b) By plotting $\lvert E^* \rvert$ as a function of $1/\omega$, a curve similar in shape to $E_r(t)$ is obtained; will be displaced somewhat from $E_r(t)$ curve for intermediate times as noted in (a) above.
Repeated loading:	Compression:	Compression:	Compression:	
(a) Axial loading	Axial stress, σ_d	Recoverable strain after a specific number of load applications, ϵ_R	Resilient modulus: $M_R = \sigma_d/\epsilon_R$	
(b) Flexure	Flexure: Applied load, P	Flexure: Recoverable deflection after a specific number of load applications, Δ_R	Flexural stiffness: $S = K \frac{P}{I \Delta_R}$ where K = constant depending on loading conditions	
Stiffness (according to van der Poel)	Penetration and ring-and-ball softening point of recovered *bitumin volume* concentration of aggregate, C_v		Stiffness, $S(t,T) = \sigma/\epsilon$	(a) Analogous to relaxation modulus.

* One statement of superposition principle: $\int_0^t D(t-\tau)E(t)d\tau = \int_0^t \epsilon(t-\tau)D(t)\tau = t.$

Table 2.3

Safe Maximum Deflections (after Hveem[45])

Pavement Thickness in.	Pavement Type	Maximum Permissible Deflection for Design Purposes* in.
8	Portland cement concrete	0.012
6	Cement-treated base (surfaced with bituminous concrete)	0.012
4	Bituminous concrete	0.017
3	Plant mix on gravel base	0.020
2	Plant mix on gravel base	0.025
1	Road mix on gravel base	0.036
1-1/2	Surface treatment	0.050

* Tentative.

Table 2.4

Variations of Stress and Strain in Thin and Thick Pavements

Thickness,* in.	E_1, psi	Stress, psi	Strain, in./in.
2	100,000	109.5	10.710 @ -4
2	500,000	889.3	9.767 @ -4
2	1,000,000	1509.0	7.908 @ -4
10	100,000	152.2	9.075 @ -4
10	500,000	289.0	3.007 @ -4
10	1,000,000	346.3	1.770 @ -4

* Thickness of the bituminous concrete layer above the subgrade soil.

Table 2.5

Factors Affecting Fatigue of Bituminous Mixes (after Finn[1])

Factor	Effect on Stiffness	Effect on Fatigue Life: Controlled Stress Mode	Effect on Fatigue Life: Controlled Strain Mode
Bitumen penetration	Increases with decrease in penetration	Increases	Decreases
Bitumen content*	Increases with increase in bitumen content	Increases	Decreases
Aggregate type	Increases with increased roughness and angularity	Increases	Decreases
Temperature**	Increases with decreasing temperature	Increases	Decreases
Void content†	Increases with decrease in voids	Increases	———
Aggregate gradation	Increase from open to dense gradation	Increases	Decreases

* Within reasonable limits above laboratory optimum bitumen content, as determined from stability tests.

** Approaches upper limit for temperatures below freezing.

† No significant amount of data; however, seems reasonable on the basis of stiffness modulus effect and data obtained in controlled stress tests.

Table 2.6

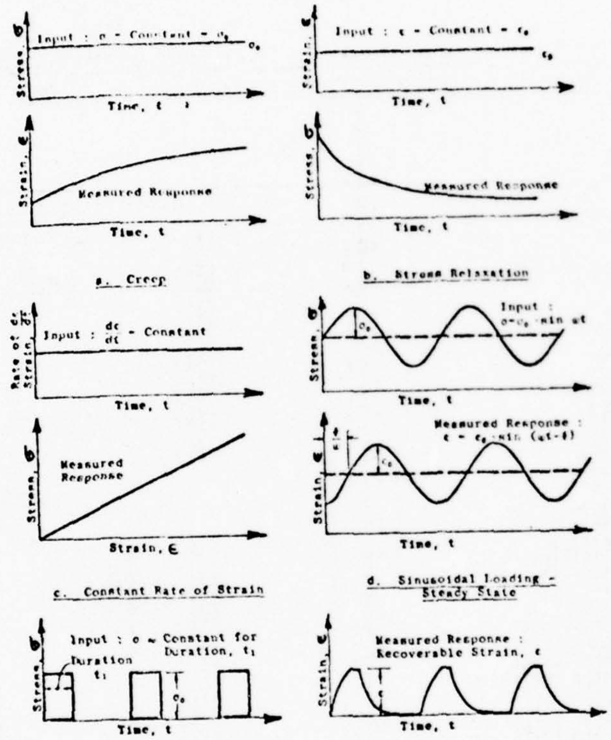
Increase in Strength of Tar Produced by Adding a Fine Slate Filler (after Rigden and Lee[95])

Percent by Weight Filler in Mixture	Tensile Strength, kg/cm^2, at Cited Temperature			Flash Point °C
	0° C	-5° C	-14° C	
0	9	6.5	5.5	-1
10	16	--	--	-2.5
20	--	15	13.5	-1
30	21	19.5	18.5	-1.5
40	26.5	22.5	22	0
50	28.5	--	--	+2

Table 2.7

Stiffness and Breaking Strength of Bituminous Base Materials (after Heukelom and Klomp[80])

Mixture Composition	Temperature °C	Dynamic Modulus kg/cm^2	Breaking Strength kg/cm^2	Strain at Break 10^{-4} in./in.
Gravel, sand, and 50 pen. asphalt cement	+10	66,000	95	2,000
Gravel, sand, and 70 pen. asphalt cement	+10	57,000	95	2,100
Gravel, sand, and 90 pen. asphalt cement	+10	50,000	100	2,700
Gravel, sand, and 110 pen. asphalt cement	+10	36,000	90	7,500
Gravel, sand, and 90 pen. asphalt cement	-10	125,000	75	1,100
Gravel, sand, and 90 pen. asphalt cement	0	85,000	90	1,400
Gravel, sand, and 90 pen. asphalt cement	+10	50,000	100	2,700
Gravel, sand, and 90 pen. asphalt cement	+20	23,000	85	9,000
Gravel, sand, and 90 pen. asphalt cement	+30	10,000	65	13,000
100% sand and 90 pen. asphalt cement	+10	50,000	85	2,700
60% sand, 40% gravel, and 90 pen. asphalt cement	+10	70,000	80	2,300
40% sand, 60% gravel, and 90 pen. asphalt cement	+10	80,000	85	2,300



REPEATED LOAD TESTS

Figure 2.1. Types of loading to measure stiffness characteristics of bituminous mixtures (after Finn[1])

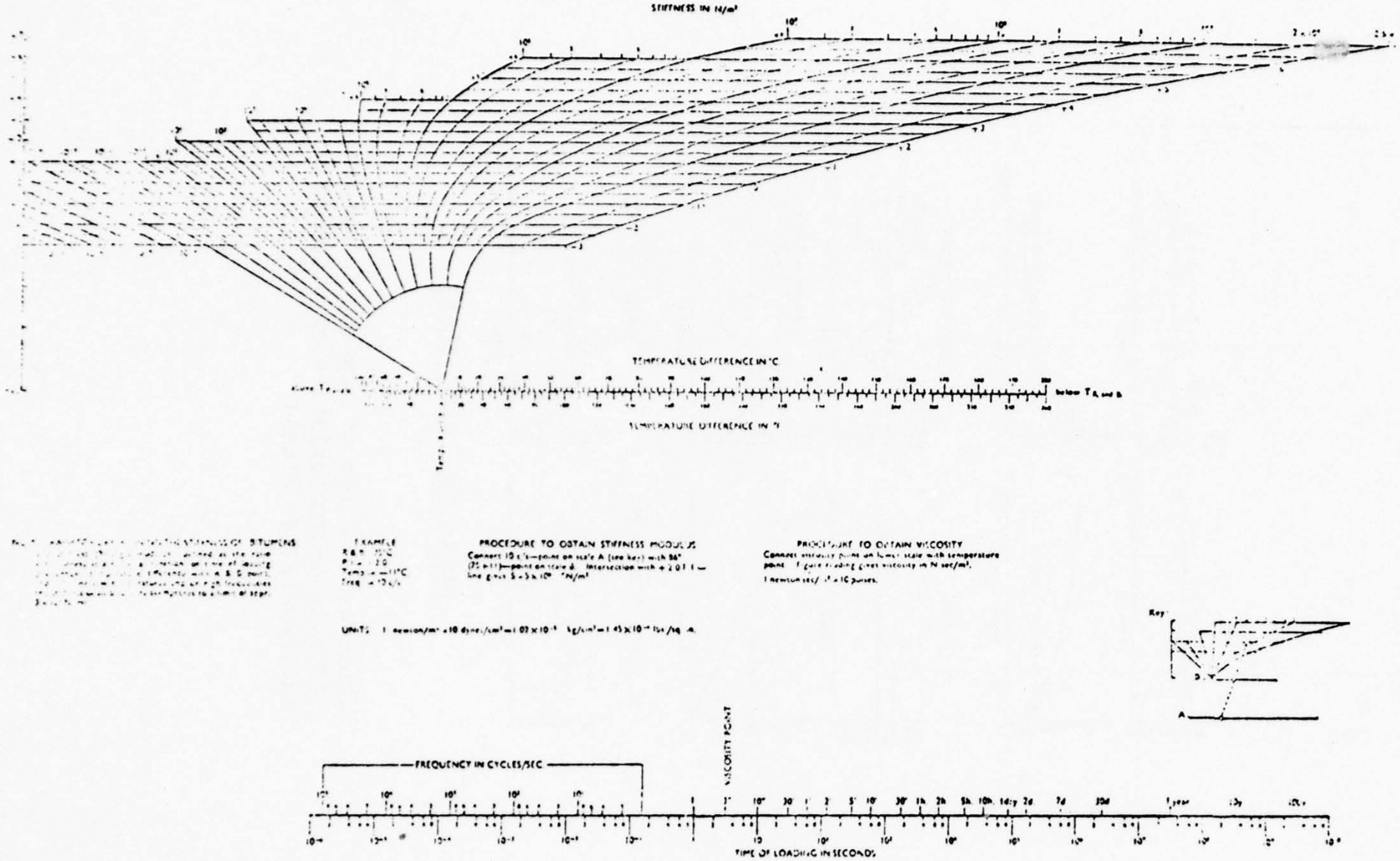


Figure 2.2. Nomograph for determining the stiffness of bitumens (after Van der Poel[11])

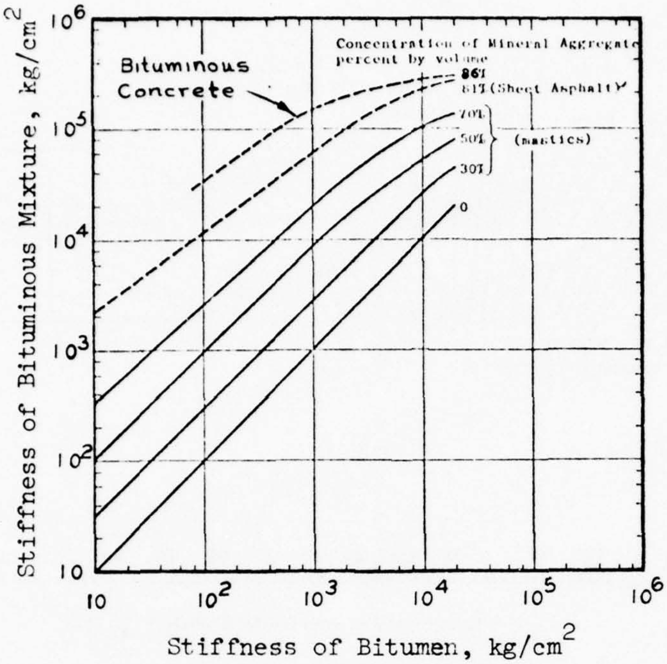


Figure 2.3. Relation between stiffness of bitumen and stiffness of bituminous mixture (after Van der Poel[11])

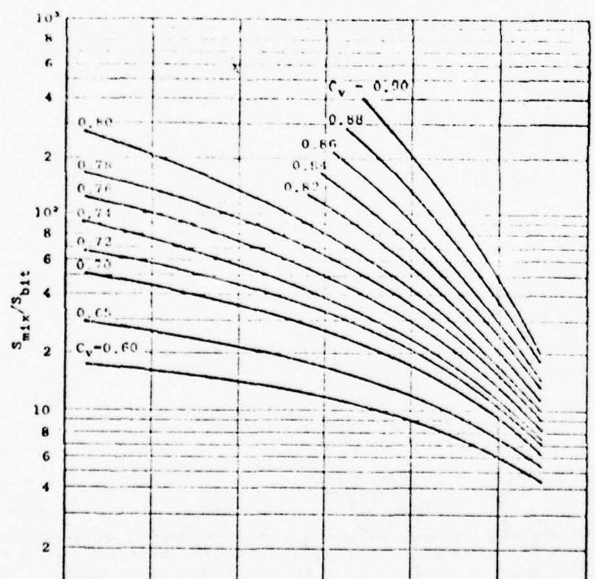


Figure 2.4. S_{mix}/S_{bit} as a function of S_{bit} and C_v (after Heukelom and Klomp[12])

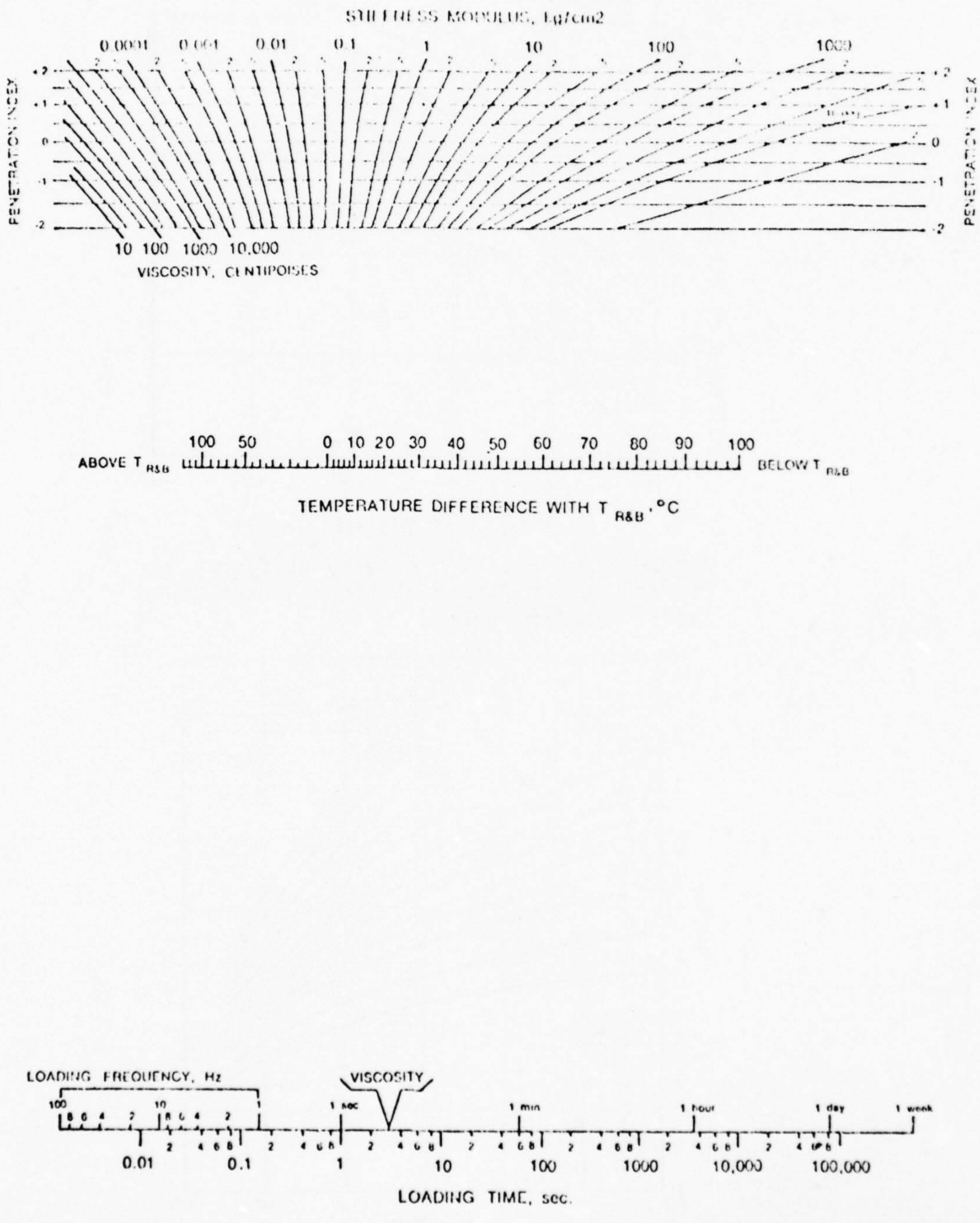


Figure 2.5. Nomograph for predicting the stiffness modulus of asphaltic bitumens (after Heukelom and Klomp[12])

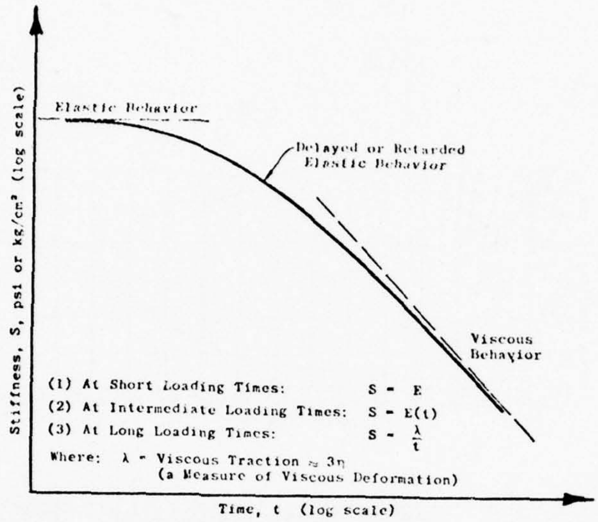


Figure 2.6. Idealized dependence on time of loading of the stiffness (stress/strain) characteristics of a bituminous material subjected to an axial tensile stress (after Finn[1])

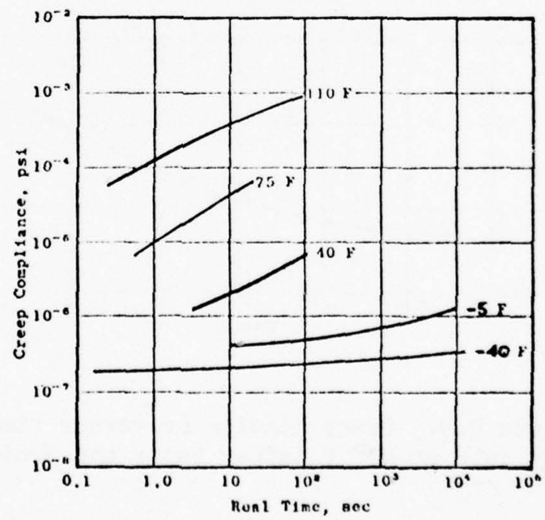


Figure 2.7. Creep compliance versus time at five temperatures for bituminous concrete (after Monismith et al.[36])

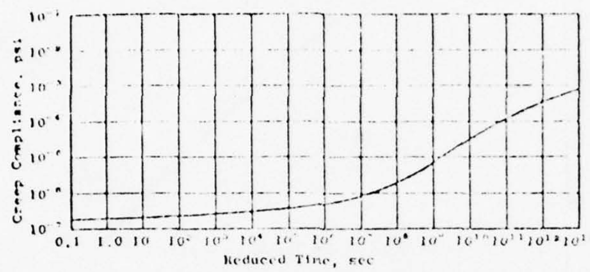


Figure 2.8. Master creep compliance curve at a reference temperature of -40° F (after Monismith et al.[36])

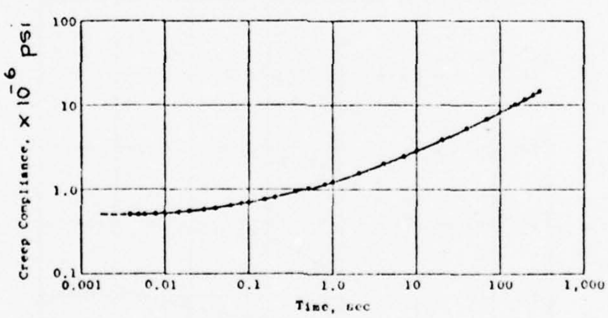


Figure 2.9. Creep compliance versus time from test data at 40° F (after Secor and Monismith[35])

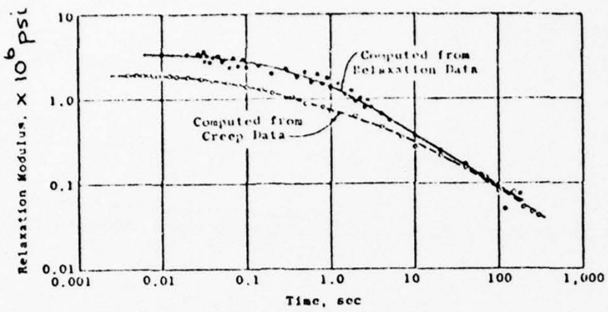


Figure 2.10. Comparison of relaxation modulus-time relationships as obtained from two sources (after Secor and Monismith[35])

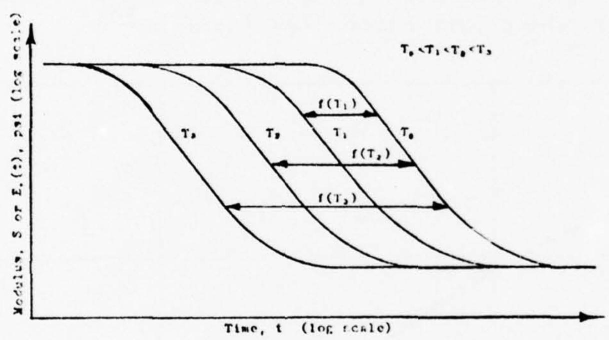


Figure 2.11. Effect of time and temperature on the stiffness modulus for a thermorheologically simple material (after Monismith et al.[36])

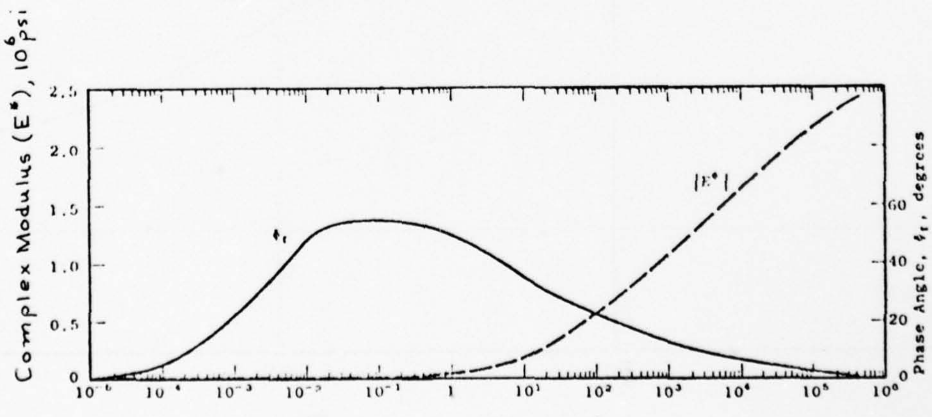


Figure 2.12. Magnitude and phase angle of complex elastic modulus E^* as function of angular frequency at 77° F (after Pagen and Ku[38])

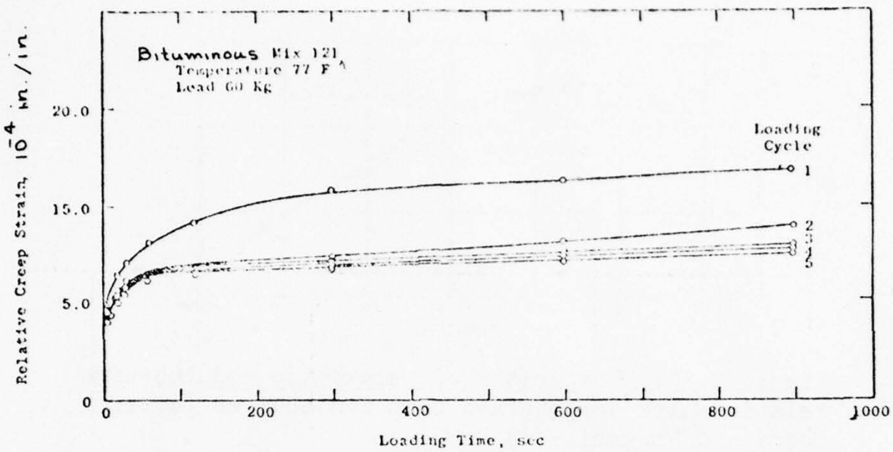


Figure 2.13. Relative creep strain versus loading time for 60-kg load (after Pagen and Ku[38])

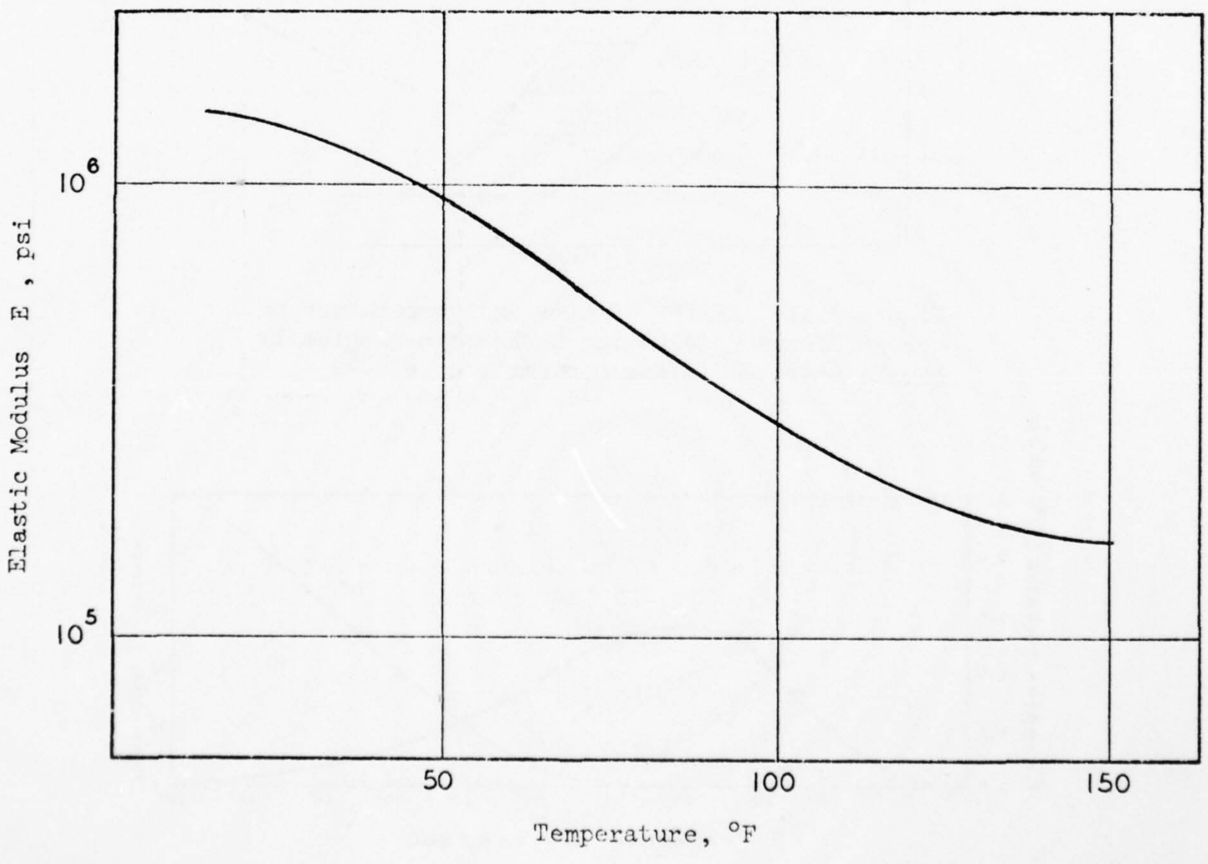


Figure 2.14. Variation of elastic modulus of bituminous concrete with temperature (after Izatt, Lettie, and Taylor[42])

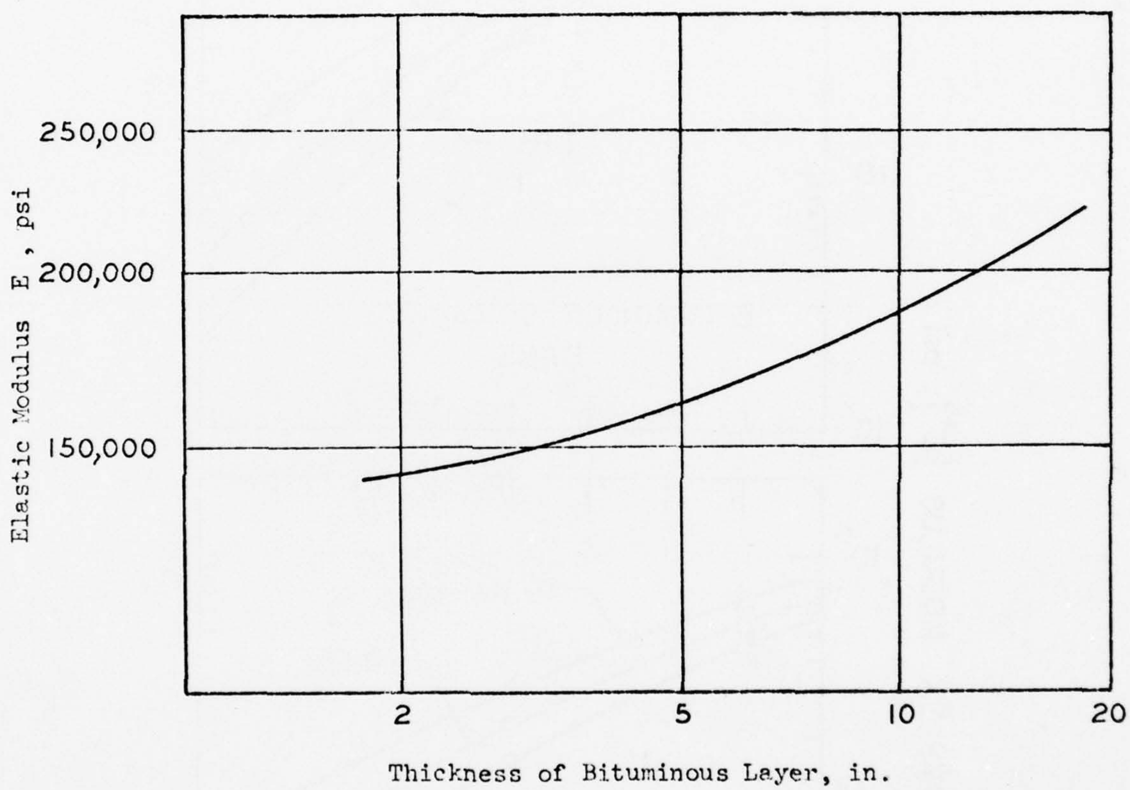


Figure 2.15. Relation of bituminous layer modulus to thickness of layer for air temperature of 95° F (after Izatt, Lettie, and Taylor[42])

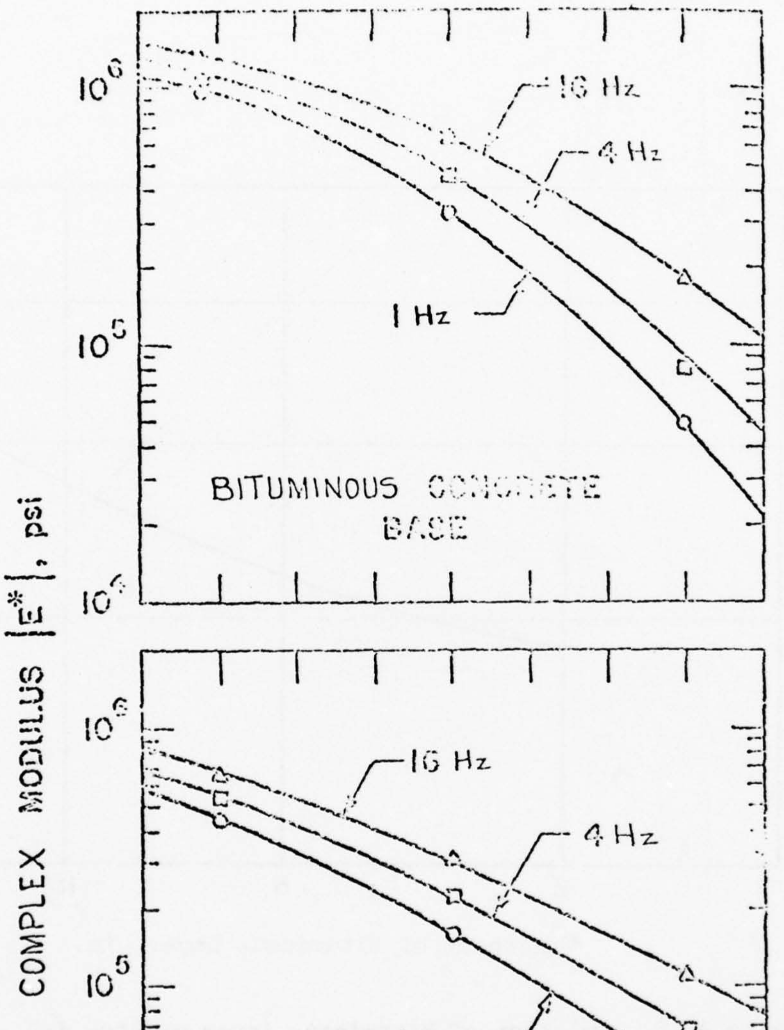


Figure 2.16. Modulus relationships for bituminous concrete base and sand-bitumen base (after Kingham and Kallas[43])

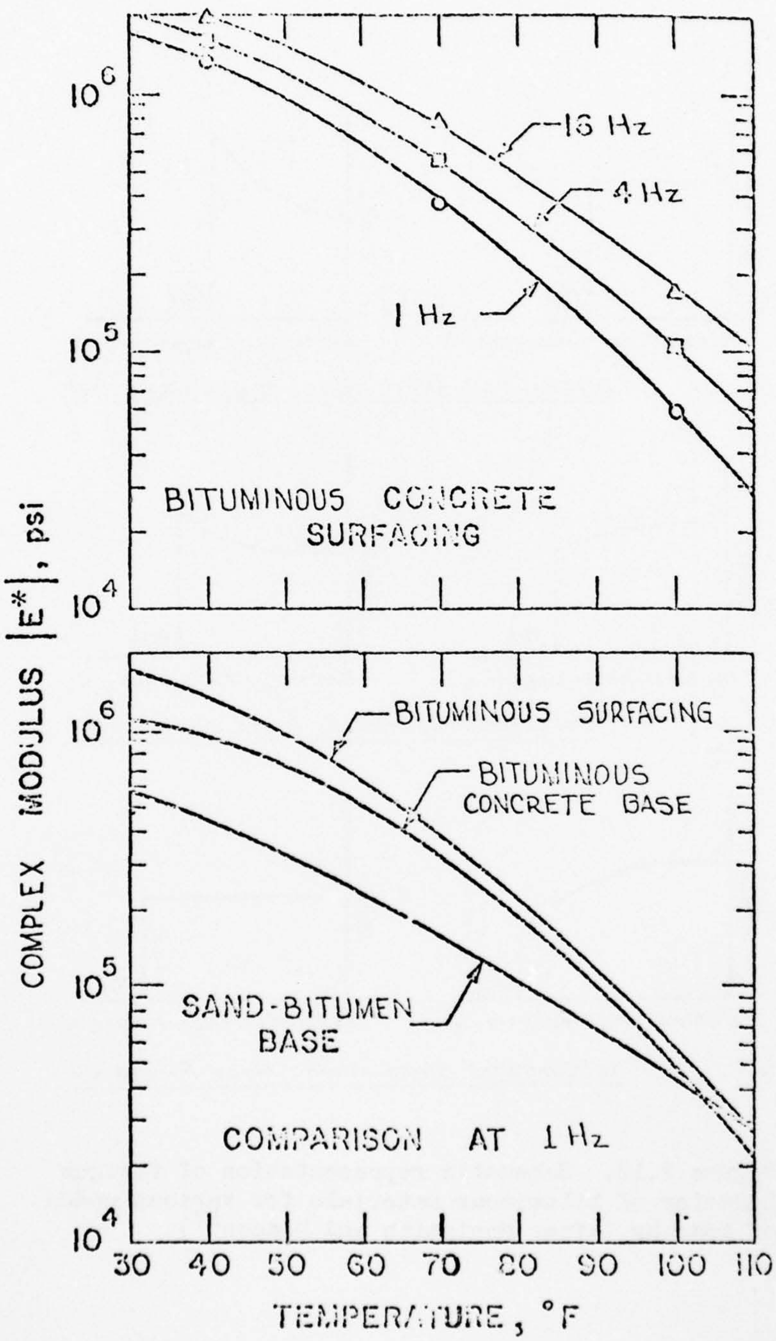


Figure 2.17. Modulus relationships for bituminous concrete surfacing and comparison curves (after Kingham and Kallas[43])

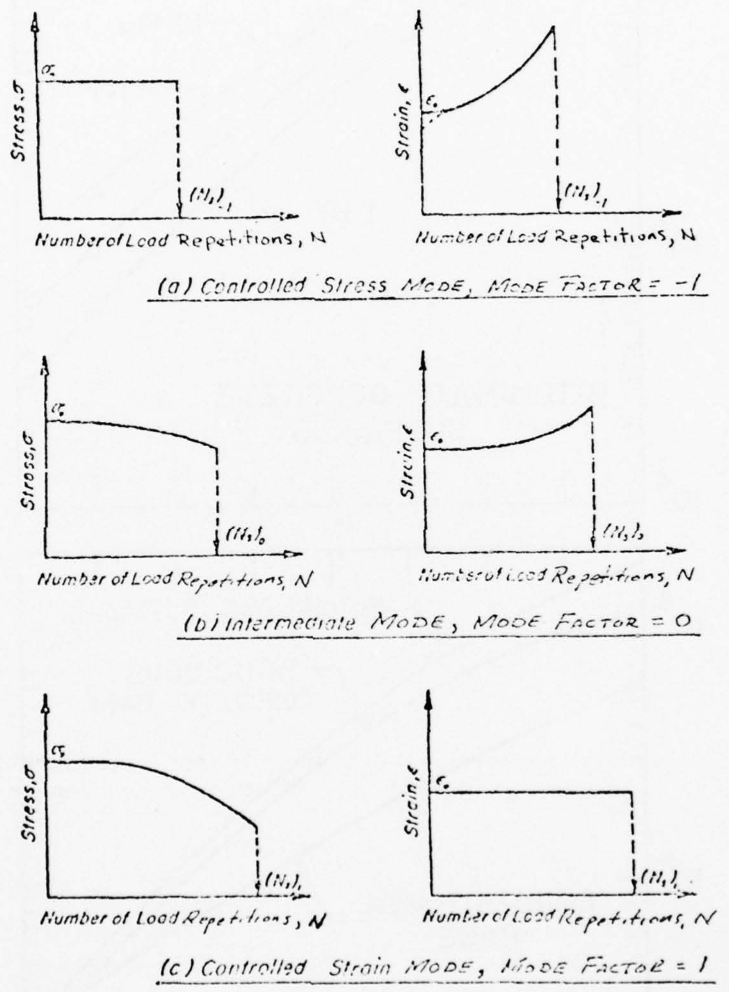


Figure 2.18. Schematic representation of fatigue behavior of bituminous materials for various modes of loading (after Monismith and Deacon[47])

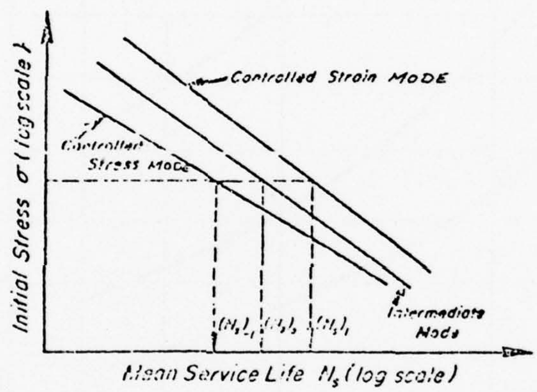


Figure 2.19. Hypothetical fatigue diagrams illustrating effect of mode of loading (after Monismith and Deacon[47])

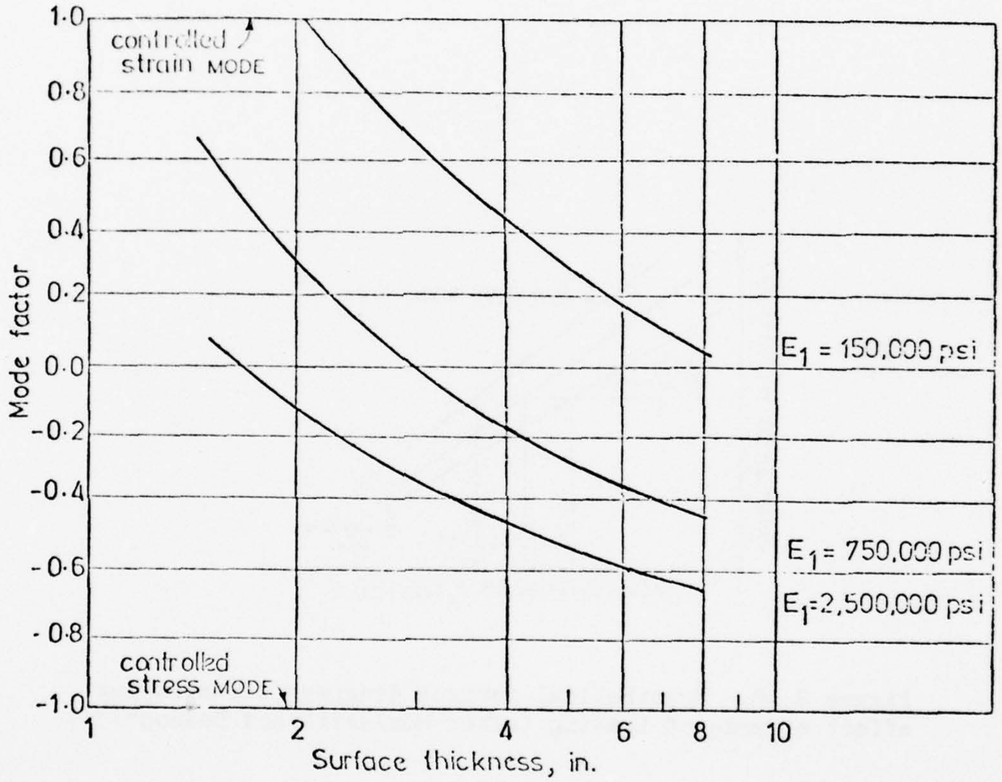
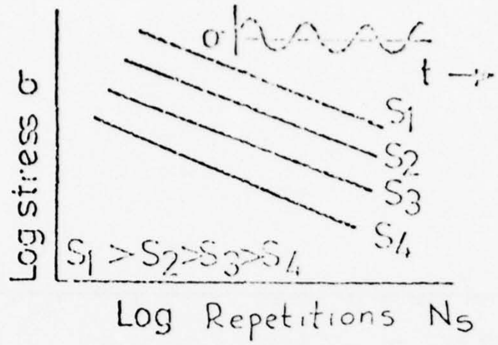
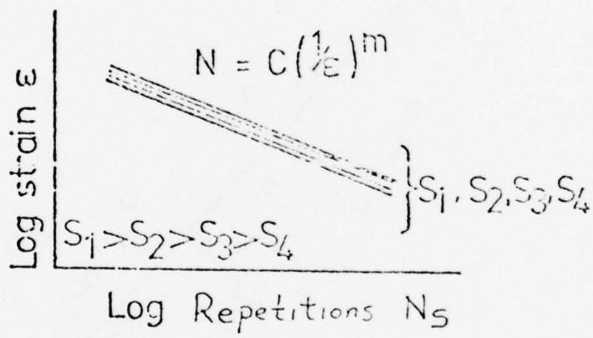


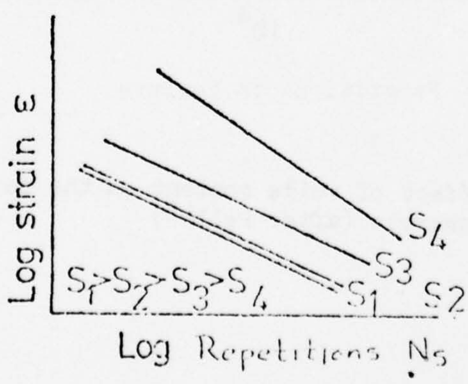
Figure 2.20. Determination of mode factor for bituminous concrete pavements (after Pell[48])



a. Controlled stress fatigue tests at different stiffnesses S_1, S_2, etc.



b. Controlled stress fatigue tests



c. Controlled strain fatigue tests

Figure 2.21. Effect of stiffness on the service life of bituminous concrete (after Pell[48])

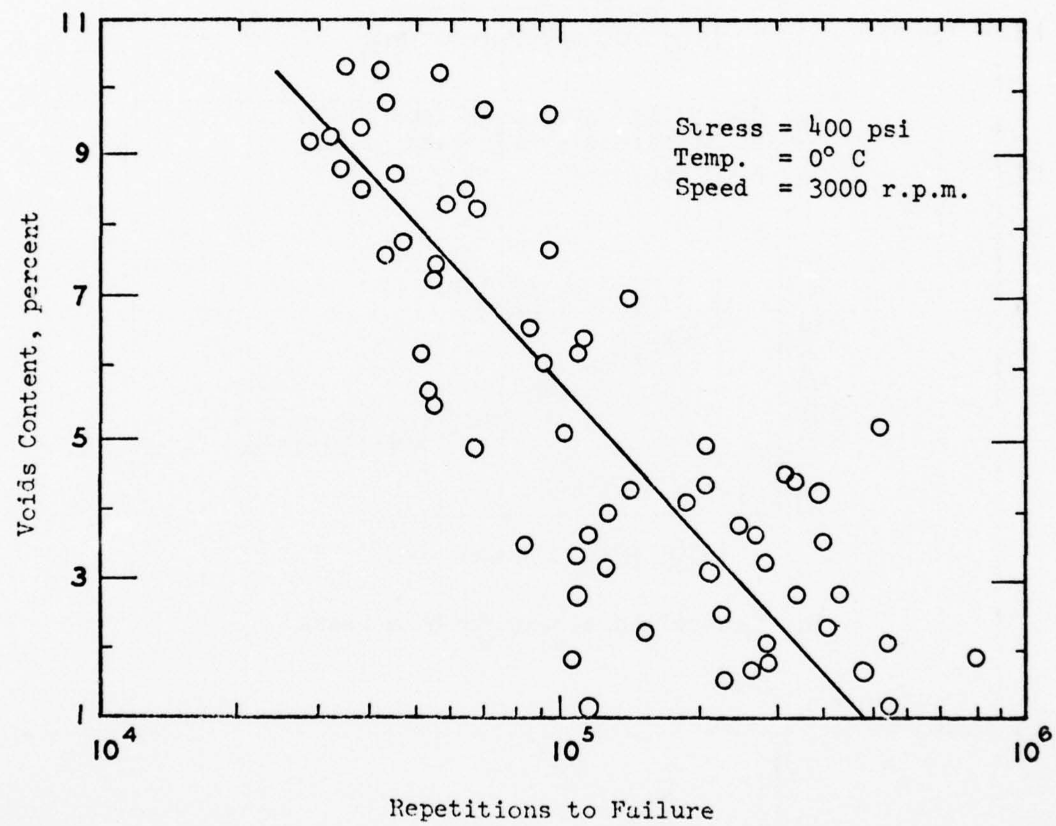


Figure 2.22. Effect of voids content on the fatigue life of bituminous concrete (after Pell[48])

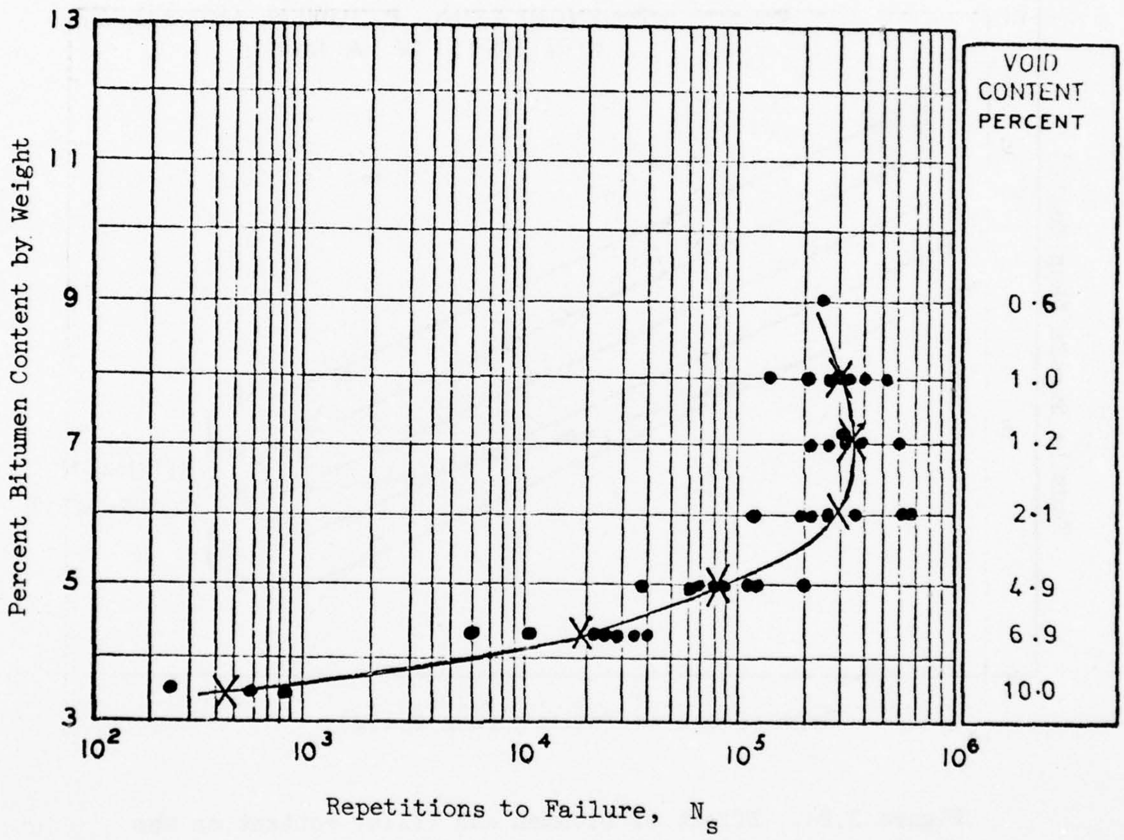


Figure 2.23. Effect of bitumen content on service life of bituminous concrete (after Pell[48])

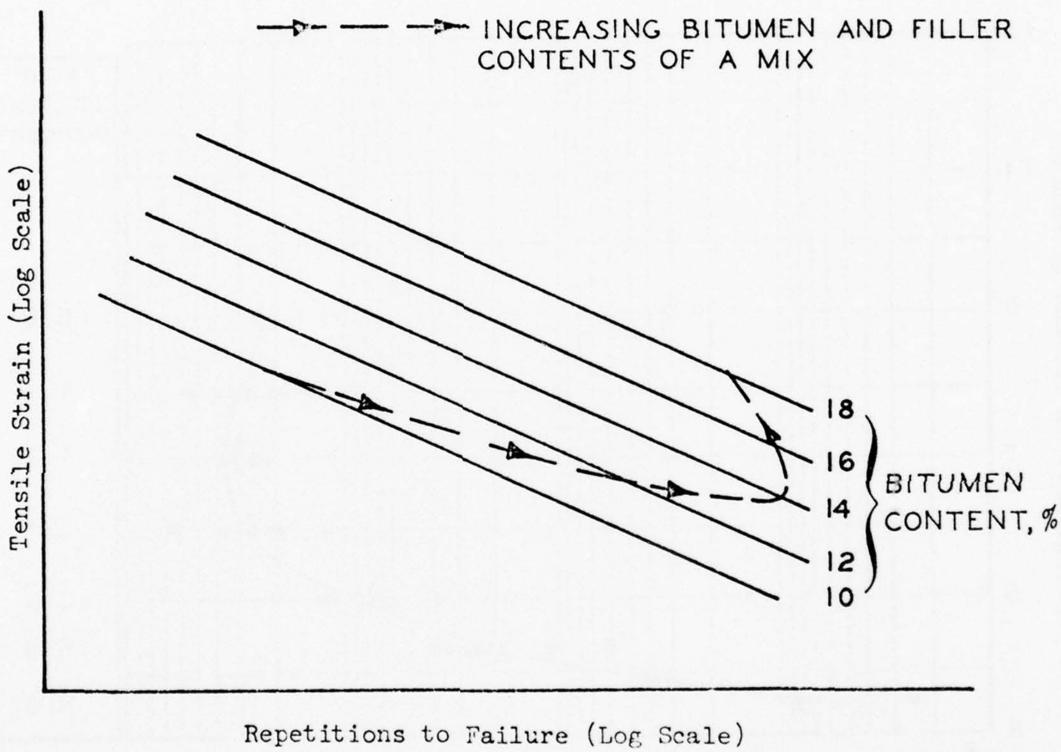


Figure 2.24. Effect of bitumen and filler content on the strain-service life relationship of bituminous concrete (after Pell[48])

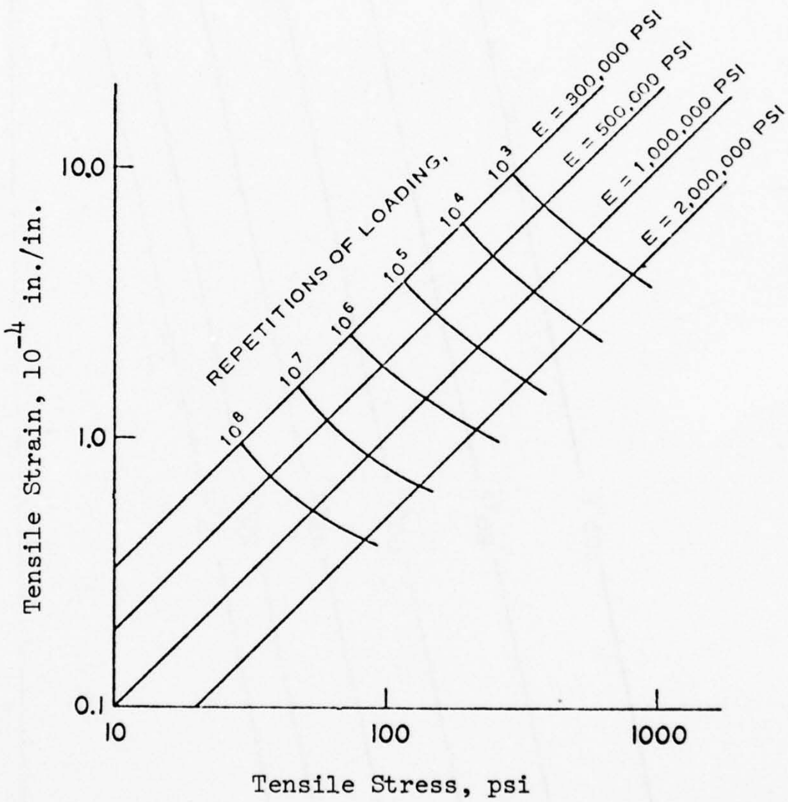


Figure 2.25. Provisional fatigue data for bituminous base course materials (after Dorman and Metcalf[63])

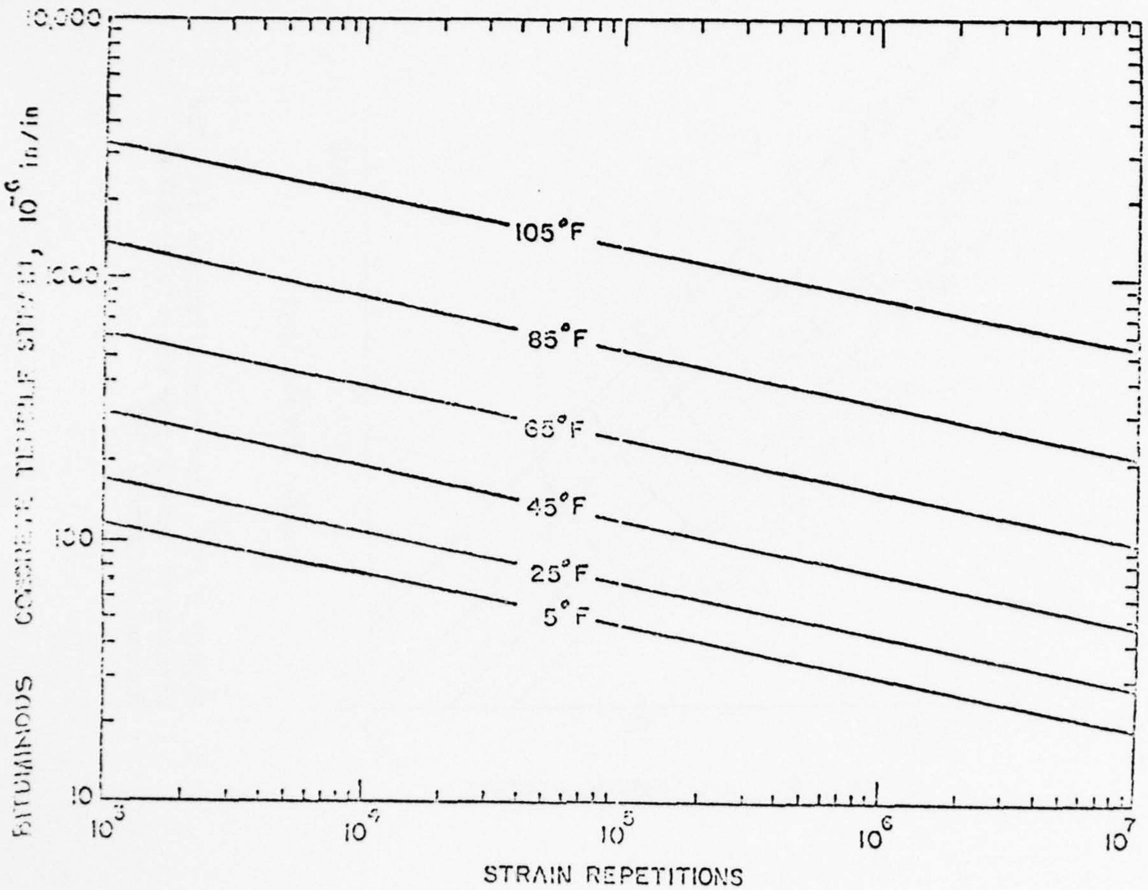


Figure 2.26. Allowable bituminous concrete tensile strain criteria (after Witczak[64])

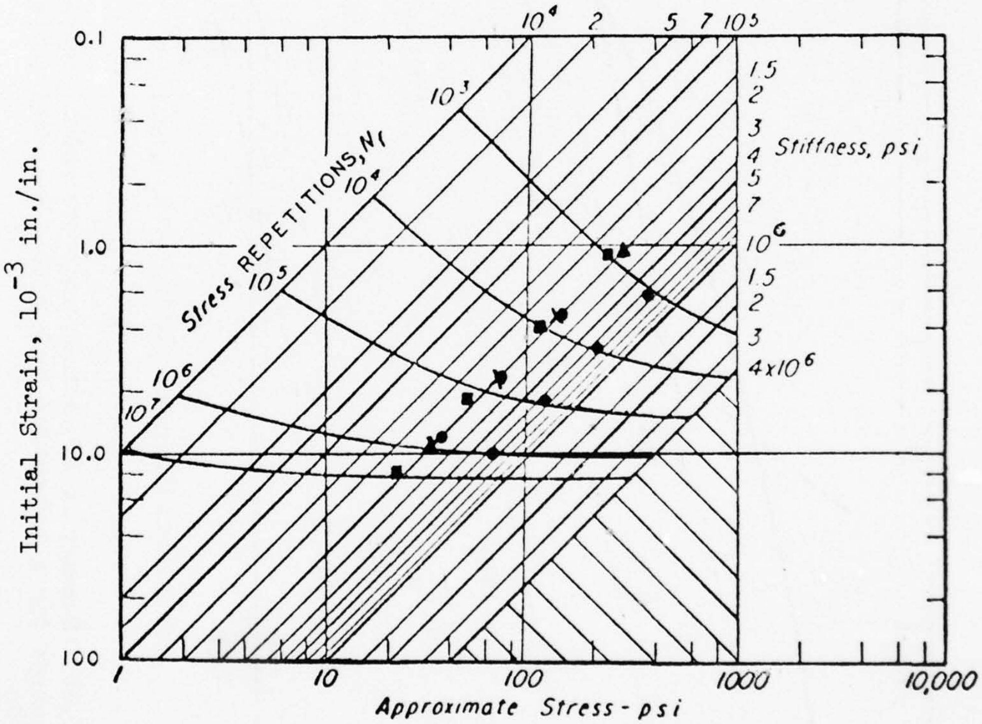


Figure 2.27. Relationship between stress, strain, stiffness, and fatigue life for California-type bituminous mixtures; approximate void content of 5 percent (after Monismith[66])

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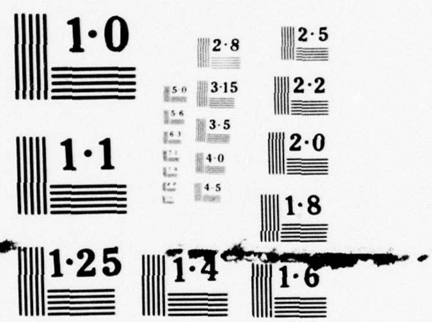

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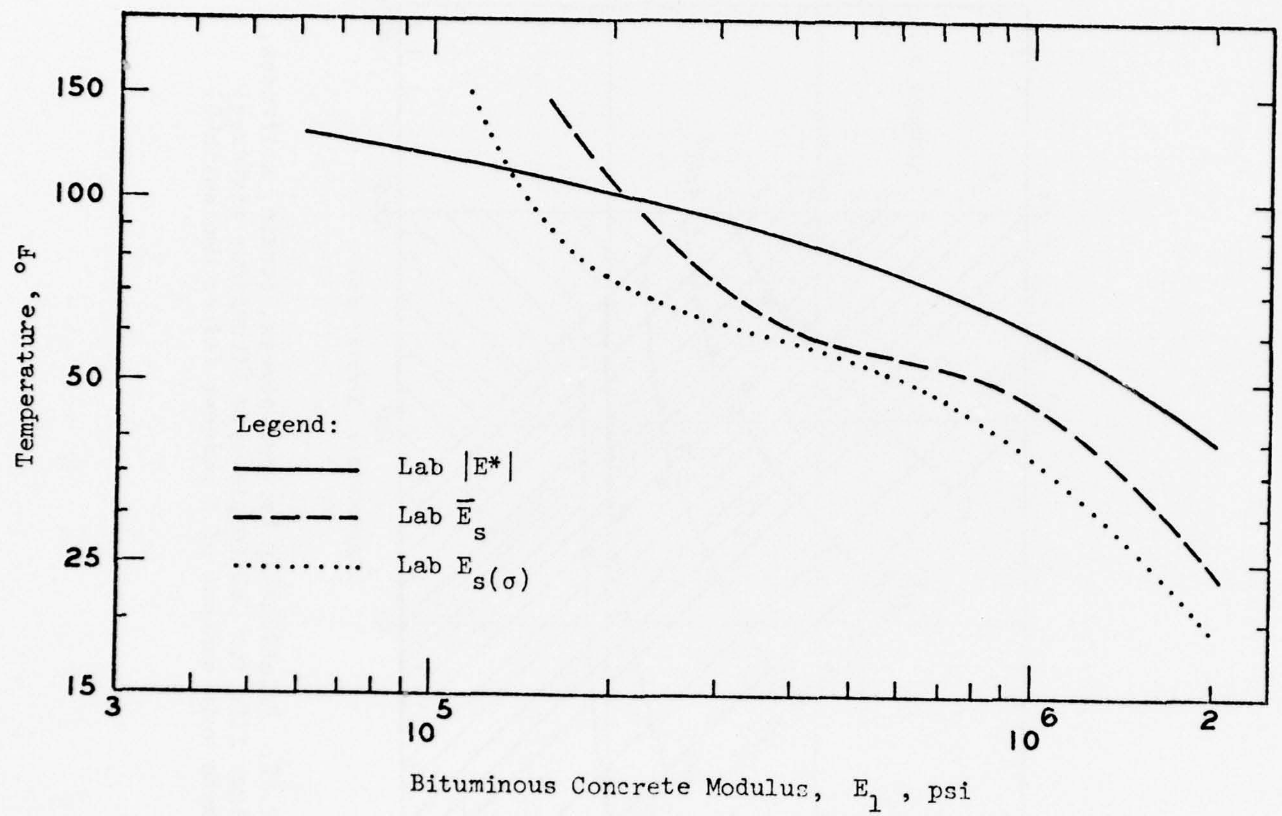

Figure 2.28. Summary of bituminous concrete moduli-temperature relationships for a bituminous base course material (after Witczak[68])

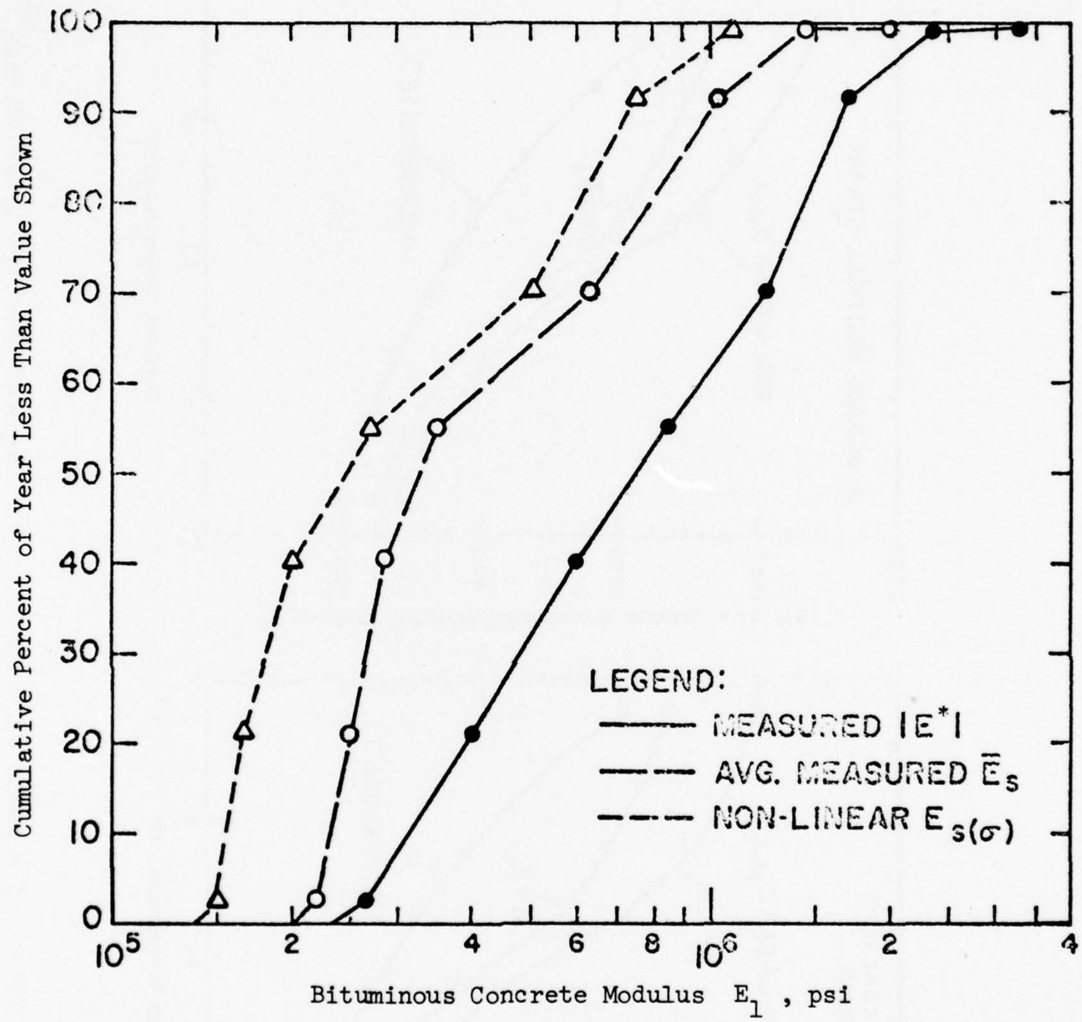

Figure 2.29. Yearly frequency distribution of bituminous concrete modulus for various methods of evaluation (after Witczak[68])

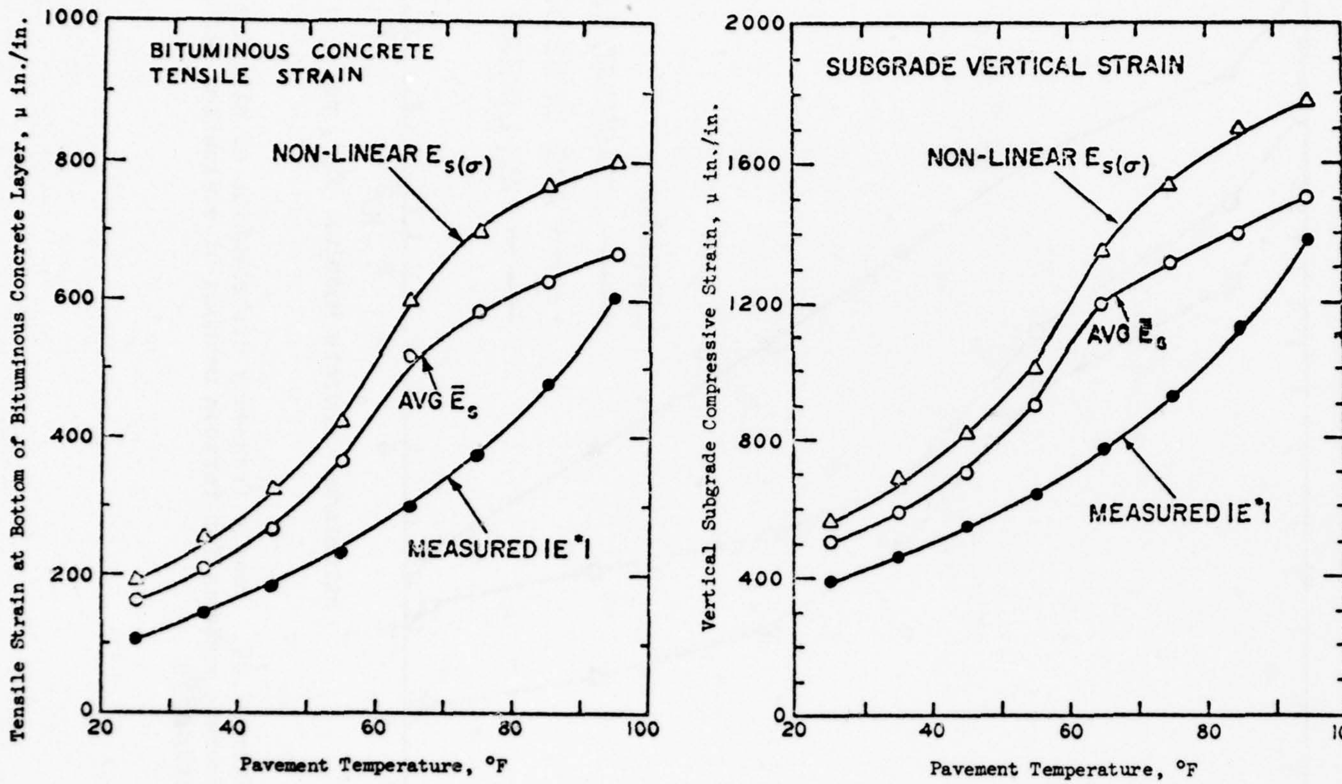

Figure 2.30. Effect of modulus relationship upon computed strains (after Witczak[68])

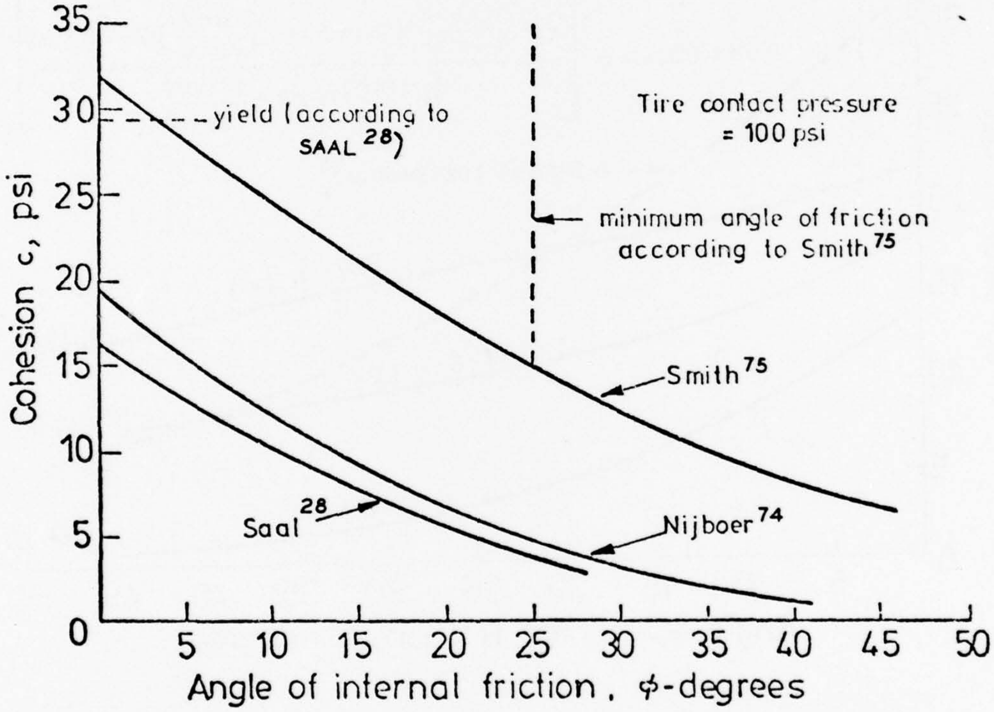

Figure 2.31. Relationship between cohesion and angle of internal friction to prevent plastic flow or overstress at a particular point in a bituminous mixture (after McLeod[76])

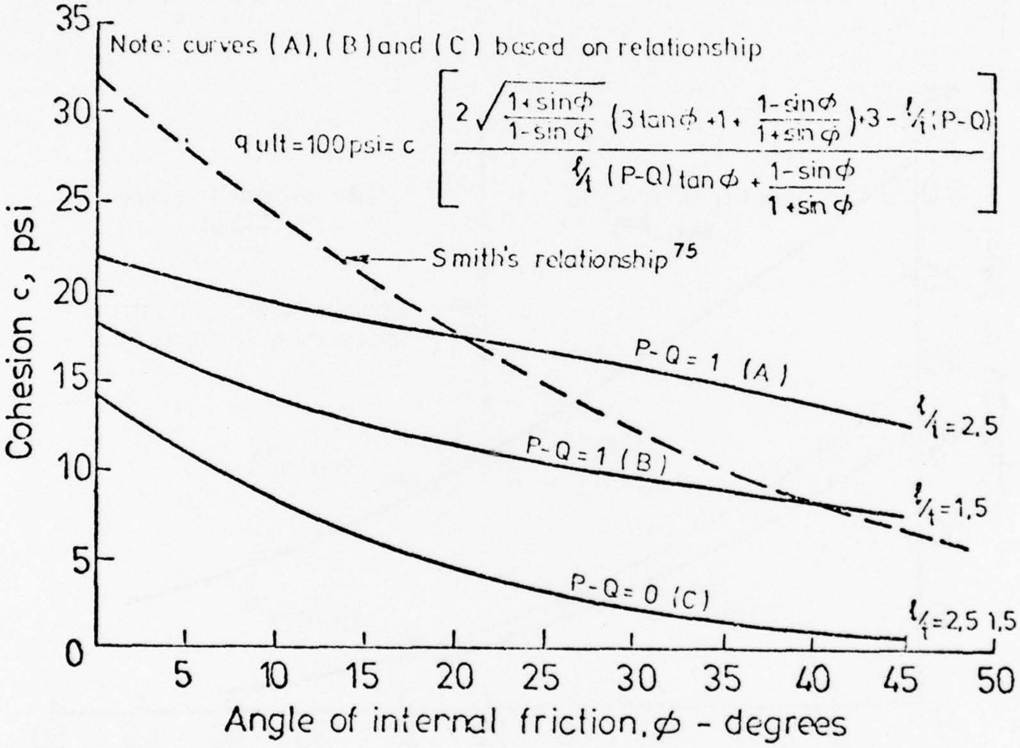

Figure 2.32. Stability curves for bituminous mixtures subjected to braking stresses (after McLeod[76])

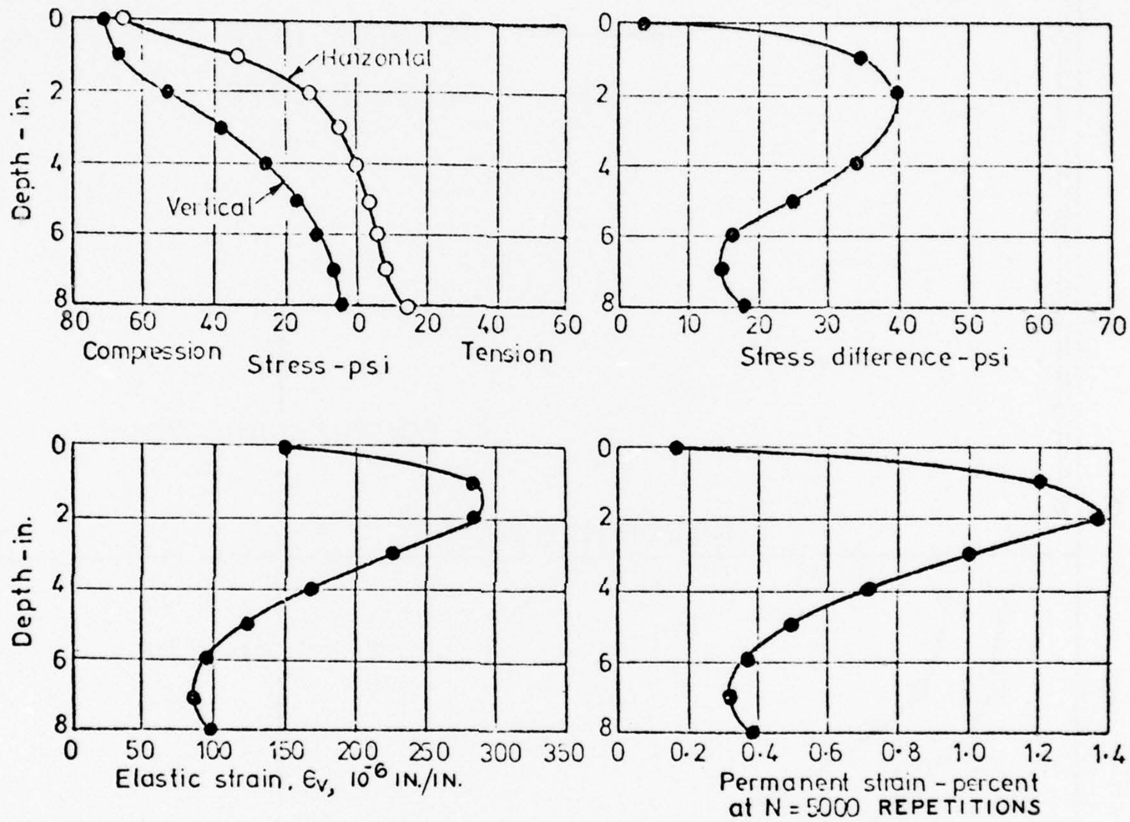

Figure 2.33. Stress and strain distributions in 8-in.-thick bituminous concrete pavement subjected to 1500-lb wheel load with 70-psi contact pressure (after McLean[70])

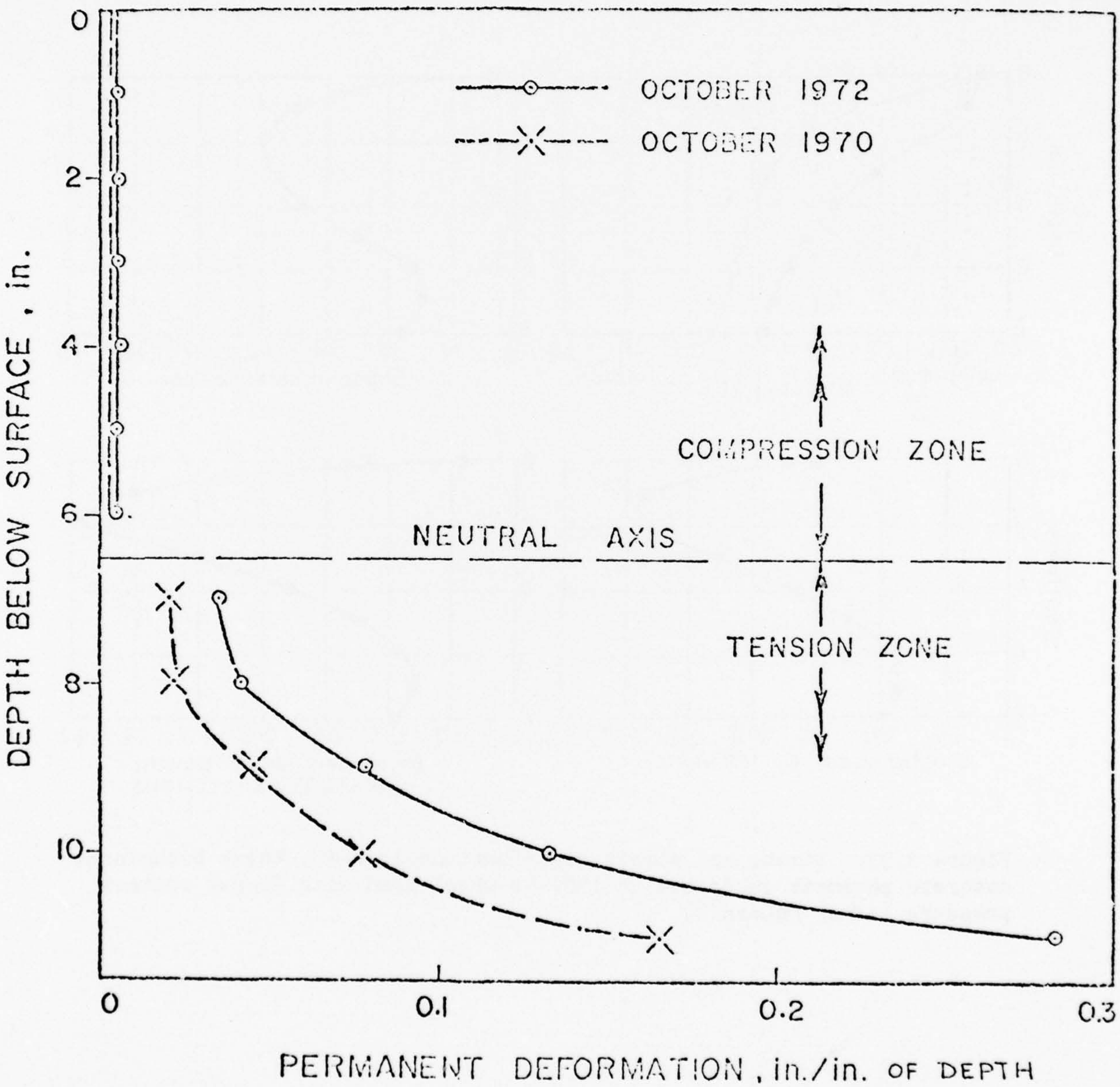

Figure 2.34. Variation of permanent deformation rate in an 11-1/2-in. bituminous concrete pavement (after Morris[88])

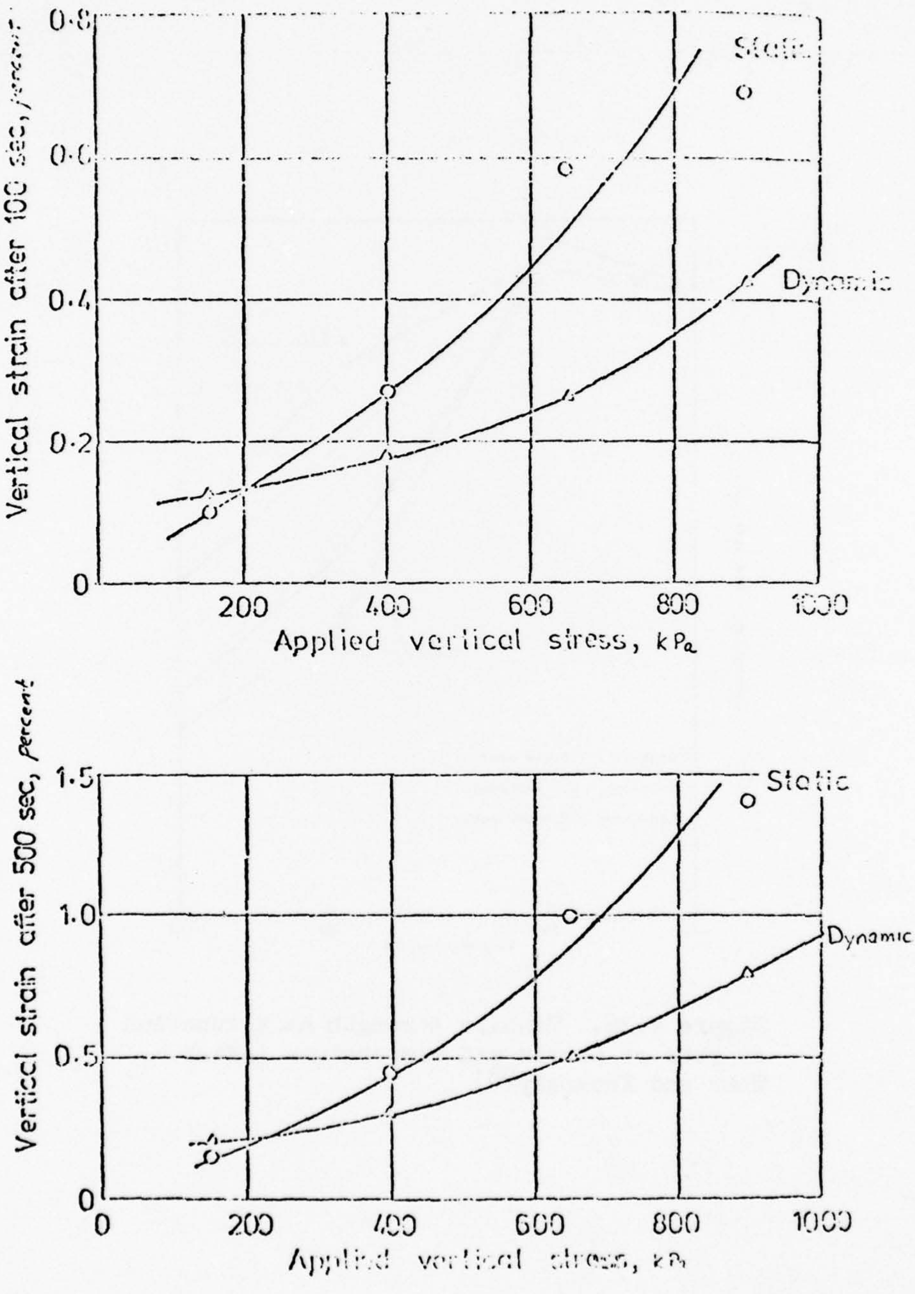

Figure 2.35. Comparison of results from dynamic tests and creep tests at 20° C

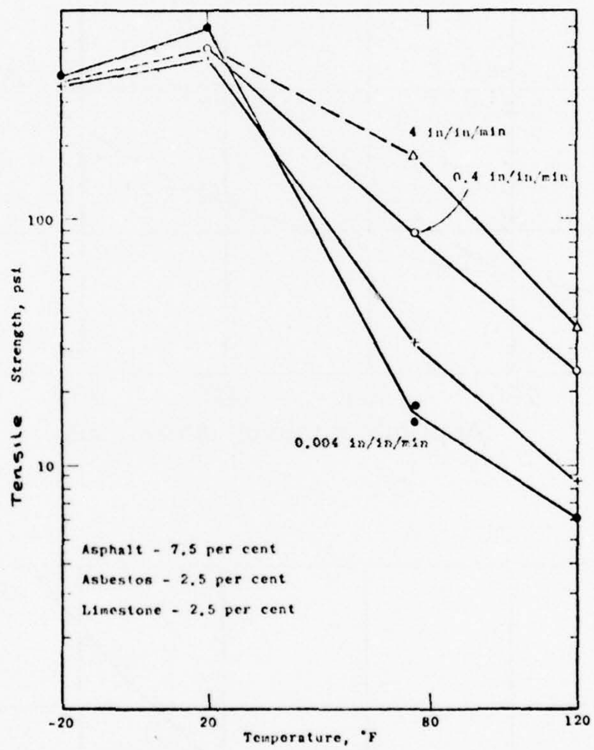

Figure 2.36. Tensile strength as a function of rate of strain and temperature (after Tons and Krokosky[98])

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NOTATION

A	Percent change in stress due to a stiffness decrease of C
A_T	Shift factor = t_T/t_o
B	Percent change in strain due to a stiffness decrease of C
c	Cohesion
C	Arbitrary but fixed percent reduction in mixture stiffness
C_v	Volume concentration = volume of compacted aggregate ÷ (volume of aggregate + volume of bitumen)
d_{ij}	Damage induced in pavement by one application of the i^{th} load while pavement is in j^{th} physical state
$d\sigma/d\varepsilon$	Slope of stress-strain curve at particular strain rate
d_1	Regression constant
$d\varepsilon_1/dt$	Rate of application of axial strain
D	Total cumulative damage; also, equivalent particle size of filler
$D(t)$	Compliance, in.^{2}/lb or cm^2/kg
$D(t,T)$	Modulus of delayed elasticity
E	Elastic modulus; also, dynamic modulus
$E(t)$	Relaxation modulus, psi or kg/cm^2
E^*	Modulus; also, complex modulus
E_c	Creep modulus
E_R	Creep modulus in flexure
$E_{s(\sigma)}$	Flexural stiffness modulus calculated from controlled stress fatigue tests
$\overline{E}_s$	Average flexural stiffness
$f(\phi)$	Function dependent on ϕ
$f(\varepsilon_v)$	Function relating permanent strain to total strain
FB	Filler-bitumen factor = volume of filler/(volume of filler + volume of bitumen)
K	Coefficient; also, a constant
K_o, K_1	Regression constants
ℓ	Length of tire tread
MF	Mode factor
n	Coefficient = slope of fatigue line; also, $n = 0.83 \log (400{,}000/S_{bit})$

n_{ij}	Predicted number of applications of aircraft load i on pavement in a particular physical state j during the design life
N_f	Fracture life = accumulated number of load repetitions necessary to completely fracture a test specimen
N_s	Service life = cumulative number of load repetitions necessary to cause failure of test specimen
N_{ij}	Number of applications of aircraft load i on pavement in a particular physical state j before failure
P	Measure of friction between tire and pavement; also, applied load
q_{ult}	Bearing capacity, psi or kg/cm^2
Q	Measure of friction between pavement and base
S	Stiffness, psi or kg/cm^2
S_{bit}	Stiffness of the bitumen, kg/cm^2
S_{min}	Minimum stiffness
S_{mix}	Stiffness of the bituminous mixture, kg/cm^2
t	Time of loading; also, temperature; also, thickness of bituminous concrete
t_o	Time required to observe the same phenomenon at the referenced temperature
t_T	Time required to observe a phenomenon at temperature T
T	Temperature
V	Void factor = 1 - (air content)$^{2/3}$
w	Frequency of loading
γ	Shear strain
δ_p	Permanent deformation
Δ_R	Recoverable deflection
ε	Strain
$\varepsilon(t)$	Measured strain as a function of time
ε_d	Delayed elastic strain
ε_e	Elastic strain
ε_o	Constant strain
ε_p	Permanent strain
ε_R	Recoverable strain
ε_v	Total strain; also, viscous strain

η	Bulk viscosity
η_{mass}	Viscosity of the mass
σ	Stress
σ_o	Constant stress
$\sigma(t)$	Measured stress as a function of time
σ_1, σ_3	Major and minor principal stresses, respectively
τ	Shear strength; also, shear (braking) stress at surface
τ_e	Initial cohesion when $d\varepsilon_1/dt = 0$
ϕ	Phase angle (or phase shift) between the stress and the resulting strain; also, angle of internal friction

CHAPTER 3: PORTLAND CEMENT CONCRETE

3.1 INTRODUCTION

The properties of concrete primarily considered in the design of portland cement concrete (rigid) pavements are Young's modulus of elasticity E, Poisson's ratio ν, flexural strength R, and the fatigue resistance of the concrete. Other concrete material properties which affect the performance of rigid pavements include aggregate gradation and soundness, cement-aggregate reaction, abrasion resistance, resistance to deicer solutions, volume change, and resistance to freeze-thaw and wet-dry cyclic changes. This chapter is chiefly concerned with the four primary properties (modulus of elasticity, Poisson's ratio, flexural strength, and fatigue resistance) and the methods used to derive these properties for design purposes.

In most current design procedures, these four primary properties are normally input as exact values even though they generally are not. Variability is normally accounted for by using low values of concrete strength or working stress and designing for the heaviest wheel loading. Safety factors are commonly used to account for concrete fatigue resulting from load repetitions. It is known that each of the primary properties of concrete is affected by the type and water content of cement; the type, grading, and size of aggregates; the proportioning of ingredients; the addition of certain additives; etc., as well as by external forces such as extreme temperatures, moisture variations, curing, and rate of loading. It is also known that the test procedures by which these properties are evaluated affect the values obtained. Although efforts have been made to standardize test procedures, differences in such things as the rate of loading, instrumentation, specimen configuration, material conditioning, etc., affect the values obtained. A number of researchers have published results of tests conducted to investigate some of the factors causing variations in these primary properties which will be reviewed in this chapter.

3.2 MODULUS OF ELASTICITY

There are several methods for determining the modulus of homogeneous elastic bodies which have been applied to concrete. Among these are static and dynamic loading in the compressive, flexural, and tensile modes; the resonant frequency method; and the pulse velocity method. While there is normally a correlation among these three methods, they do produce different results on the same concrete specimen. In addition, there are variations in the way modulus and Poisson's ratio tests are conducted involving the methods of loading and the instrumentation for measuring the applied load and resulting strains.

The modulus of elasticity in tension or compression is a constant which expresses the ratio of unit stress to unit deformation for all values of stress not exceeding the proportional limit of stress. The formula is as follows:

$$E = \frac{\text{unit stress}}{\text{unit deformation}} = \frac{\sigma}{\varepsilon} = \frac{\frac{P}{A}}{\frac{e}{L}} = \frac{PL}{Ae} \qquad (3.1)$$

where

- E = Young's modulus of elasticity
- σ = unit stress
- ε = unit of deformation
- P = applied load (axial)
- A = cross-sectional area
- e = total deformation
- L = length of specimen

This is a measure of the ability of the concrete to resist deformation. As the modulus increases, the deformation decreases for each unit stress. The modulus of concrete is often the same in tension and compression.

3.2.1 TEST PROCEDURES FOR DETERMINING MODULUS OF ELASTICITY

<u>3.2.1.1 Static Modulus (Compressive).</u> Figure 3.1 shows a typical

stress-strain curve for a concrete specimen loaded in compression. The modulus can be calculated from: (a) the initial tangent, (b) a tangent at any point on the curve, (c) a secant modulus, or (d) a chord modulus. Both CRD-C 19[1] and ASTM C 469[2] require that the modulus be calculated from stress at 50μ in./in. to stress at 40 percent ultimate stress, and this is called the chord modulus. Calculation by the different methods, however, usually produces different moduli. An example, shown below, was calculated from a typical stress-strain curve for a 6- by 12-in. concrete cylinder.

	Stress, psi	Modulus, 10^6 psi
Initial tangent	0- 250	6.25
Tangent	250- 500	4.55
Secant	0-1000	4.55
Chord	250-1000	4.17

In static modulus tests, the strain of the specimen under stress is usually measured with dial gages; however, either embedded electric gages or strain gages attached to the surface of the specimens may be used. Strain measurements will vary to some degree, depending on the method used to determine strain.

3.2.1.2 Static Modulus (Tensile). Although there have been many attempts to develop a satisfactory test procedure to apply direct tension to a concrete test specimen, to date there is no universally accepted method. The major problem has been to devise a method for clamping or otherwise holding the test specimen without inducing concentrated stresses which influence the test results. Methods using a reduced cross-sectional area or bonding the ends of the concrete specimen to the end caps have generally proven most satisfactory. Good results using the latter of the above methods have been reported in Kadlecek et al.[3] As with the compression test, a stress-strain curve is generated by applying direct tension to the test specimen and the modulus is computed. The modulus values obtained from the direct tension test seem to agree with those obtained from the compression test;[4] however, because of the end clamping problems, there is generally a higher variability in the measured values.

3.2.1.3 Statis Modulus (Flexure). There are two types of flexural tests conducted on concrete: the simple beam with third-point loading (CRD-C 16-66[1]) and the simple beam with center-point loading (CRD-C 17-69[1]). The difference between the two test methods is the loading arrangement. The third-point loading test is commonly used for pavements. During the test, the deflection of the loaded beam is measured and the modulus of elasticity in flexure is calculated from

$$E = \frac{23PL^3k}{1296DI} \tag{3.2}$$

where

P = applied load

L = span length

k = Pickett's correction for shear (third-point loading)
$= \left[1 + \left(\frac{216}{1.15} + \frac{27}{23}\nu\right)\left(\frac{h}{L}\right)^2\right]$

ν = Poisson's ratio

h = height of beam

D = deflection under load P

I = moment of inertia ($wh^3/12$ for rectangular sections)

w = width of beam

There is no equation formulated for determining the modulus of elasticity under center-point loading.

3.2.1.4 Dynamic Modulus (Resonant Method). To obtain the resonant modulus (CRD-C 18[1]), specimens are vibrated or driven electrically, and the resonant frequency or frequency producing the maximum amplitude is detected by a pickup circuit and is read from a variable-frequency audio-oscillator. The modulus is proportional to the square of the resonant frequency, and the formula is as follows:

$$E = CWn^2 \tag{3.3}$$

where

C = factor which depends on the shape and size of the specimen, the mode of vibration, and to some extent, Poisson's ratio

W = weight of specimen

n = resonant frequency

Authorities differ as to the correct value of C , and this appears to be a source of variation. The advantage of the resonant modulus is that the procedure for determining it is nondestructive, and thus the value can be used as a measure of progressive changes in the strength of the concrete. The resonant modulus is used extensively in evaluating the resistance of concrete to such action as freeze-thaw and wet-dry cycles or chemical attack.

3.2.1.5 Dynamic Modulus (Pulse Velocity Method). The pulse velocity method can be used to evaluate the dynamic properties of concrete by measuring the velocity of a pulse traveling through concrete. The pulse method was developed because the earlier developed resonant frequency techniques were not suitable for testing concrete in situ. It was appreciated that the techniques developed for laboratory specimens could not readily be extended to use in field testing because of the difficulty and the danger of vibrating a structural member at resonance and because of the complexity of the computations which would be required to convert the resonant frequency to some significant quality of the concrete.

The distinct advantage of the pulse velocity method over the resonant frequency method is that it need not be confined to specimens of regular shape. Concrete in place can be tested by this method. Pulse velocity measuring devices can be divided into three classes: single-blow devices, ultrasonic devices, and repetitive-blow devices. The equipment and measuring techniques have been described in detail in Whitehurst.[5]

The single-blow device is an electronic interval timer. In the procedure, a hammer blow is applied to the concrete and the pulse velocity is measured. One of the instruments classified as ultrasonic devices is known as a "soniscope" and was originally developed as a crack-detecting device for use on monolithic structures. It has proved quite satisfactory for general use in determining the dynamic properties of concrete in place. The soniscope has been used extensively

to test dams, navigation locks, highways, bridges, and buildings, as well as laboratory specimens. It has also been used on materials other than concrete, including wood poles and stabilized soil mixtures. With respect to concrete, it has been used over path lengths ranging from 2 in. to greater than 50 ft. The other instrument used for measuring pulse velocities through concrete was developed at the Road Research Laboratory, England. It is called an "ultrasonic concrete tester." The device is in many respects similar to the soniscope. Published reports indicate that this equipment has been used primarily in the laboratory for precise studies of the variation in quality of concrete specimens. It has, however, been used in the field on walls and other structures to determine the uniformity of quality of the structural members. It has also been used in the laboratory to study the formation of cracks in a specimen as it undergoes tension or compression testing. The repetitive-blow devices combine the cathode-ray oscilloscope feature of the soniscope with the physical-blow techniques of the electronic interval timer.

The modulus of concrete can be computed using the pulse velocity data by the formula

$$E = dV^2 \frac{(1 + \nu)(1 - 2\nu)}{1 - \nu} \tag{3.4}$$

where

d = density

V = compressional wave velocity

ν = Poisson's ratio

A major difficulty in the application of the formula is the necessity for knowing Poisson's ratio. The resonant method is only slightly affected by Poisson's ratio. As pointed out by Philleo,[6] a change in the ratio from 1/6 to 1/4 increases the computed value of E less than 2 percent in the resonant method; but in the pulse method the same change in Poisson's ratio can be determined by the soniscope only if the velocity of the shear wave or of the Rayleigh wave can be measured

in addition to the velocity of the compressional wave. In general, however, only the compressional wave can be identified with sufficient accuracy to obtain a reading.

3.2.2 COMPARISON OF METHODS FOR DETERMINING YOUNG'S MODULUS OF ELASTICITY OF CONCRETE

Philleo[6] discussed the elastic response of concrete to static, resonant frequency, and pulse velocity tests and presented the results shown in Tables 3.1 and 3.2. These data were developed by the Portland Cement Association Laboratories and show that, except for cylinders with a low modulus of elasticity, the resonant modulus agrees well with the static modulus, but there is a tendency for the resonant modulus to become larger than the static modulus as the modulus decreases. The pulse velocity modulus, however, was significantly lower in every case than the static modulus. Philleo came to the following conclusions:

> The dynamic methods should not be expected to check the static methods exactly for two reasons. First, the dynamic methods deal with almost purely elastic effects, whereas static measurements are complicated by inelastic deformations; and secondly, the various methods are affected by the heterogeneity of concrete in different ways. For predicting the deflections of structural members, no particular value of Young's modulus is adequate since the deflection is a function of the duration as well as the magnitude of the load. The difference between the resonant modulus and a particular secant modulus may be thought of as a measure of the plastic deformation occurring during the loading period.
>
> It is doubtful whether any practical use can be made of values of Young's modulus computed from pulse velocities. If the paste and aggregate differ in elastic properties, the formula used for calculating the modulus is misapplied, and the results are likely to be misleading. Since the pulse velocity is a characteristic of the concrete independent of size and shape of specimen, the velocity itself would appear to be a more significant property than something calculated incorrectly from it. Thus the pulse technique does not appear promising for comparing, even on a relative basis, elastic properties of concretes containing different aggregates. It does, however, have a promising field in studying the setting and hardening of green concrete and the durability of concrete at later ages. Changes with time of the pulse velocity in a single piece of concrete must surely be significant.

Mather,[7] in discussing Philleo's paper, presented the data shown in Figure 3.2. Mather reported laboratory investigational work on 193 different concrete mixtures, 109 at a water-cement ratio of 0.8 by weight and 84 at a ratio of 0.5. At 180 days age, a 3- by 6-in. cylinder from each mixture was tested for chord static modulus (250 to 1000 psi) in accordance with CRD-C 19,[1] and two 3-1/2- by 4-1/2- by 16-in. beams were tested for fundamental transverse frequency in accordance with ASTM C 215.[2] Figure 3.2 gives the average values of each of the 193 mixtures. It should be noted that the dynamic moduli were about 1×10^3 psi higher than the corresponding static moduli. Mather stated that this difference could be primarily the result of differences in the size and shape of the specimens tested, the particular static moduli computed, or both. It should be noted that Philleo compared the initial tangent and secant static moduli with the pulse and resonant moduli, while Mather used the chord static modulus (250 to 1000 psi) and the resonant modulus.

Shideler,[8] in investigating lightweight aggregate concrete for structural use, conducted both secant static modulus and resonant modulus tests on moist- and dry-cured lightweight concrete and sand-gravel concrete varying in compressive strength from 3,000 to 10,000 psi. He found that resonant (dynamic) modulus values of moist-cured lightweight concrete were about 350,000 psi greater than static modulus values. As the concretes dried, this difference diminished. The resonant modulus of sand-gravel concrete was about 1,500,000 psi greater than the static modulus for moist-cured specimens and about 750,000 psi greater than the static modulus for dry-cured specimens.

Whitehurst[9] surveyed the results of a quarter century of efforts at correlating the resonant modulus with one of the several static modulus values of concrete. He concluded that, if resonance tests are carefully performed and the resonant modulus computed through use of equations taking into account all appropriate corrections, the resulting values may be reasonably representative of the secant static modulus corrected for technique and size of specimen.

Batchelder and Lewis[10] reported experiments in which resonant frequency and pulse velocity tests were used to evaluate the performance of 3- by 4- by 16-in. beams as they were subjected to freezing and thawing cycles. The results are shown graphically in Figure 3.3. The values were not numerically equal for a given specimen, but showed a good correlation except for concretes that had been significantly damaged or deteriorated. The pulse velocity value was lower by about 1×10^6 psi than the resonant modulus before freezing and thawing but higher by about the same factor after freezing and thawing.

Tests were conducted in the Concrete Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES) on fibrous concrete cylinders and beams, and a comparison was made between Young's modulus of elasticity obtained by the flexure method (CRD-C 21[1]) and the dynamic (pulse velocity) method (CRD-C 51[1]). The tests were conducted on specimens varying in compressive strength from approximately 4000 to 8500 psi. There was no significant difference between the moduli obtained by the two methods. Results of elasticity in flexure tests were also compared with initial tangent static moduli on some of the above mixtures, and the flexure moduli averaged 1.7×10^6 psi higher than the initial tangent moduli.

A research program was conducted in the Concrete Laboratory at WES and reported in 1963[11] comparing the static modulus of elasticity (compressive secant) and the dynamic modulus (compressive and tensile) obtained by an impact test (dropping a known weight (force)) on a concrete specimen. Load cells were used to measure the induced stress, and strains were measured with SR-4 strain gages. The stress and strain were recorded by photographing an oscilloscope at the instant of impact. The tensile splitting modulus was obtained by attaching strain gages centered horizontally on the ends of a concrete cylinder. The dynamic modulus in compression and in tension compared very closely with the static compressive secant modulus, being slightly lower for low-strength concrete and higher for the high-strength concrete.

The abstracted data presented herein comparing methods of determining the modulus of elasticity can be summarized as follows:

a. The static modulus can vary by as much as 2×10^6 psi, depending upon whether an initial tangent, tangent, secant, or chord modulus is calculated.

b. The pulse velocity modulus is generally about 1×10^6 psi higher than the resonant modulus, which in turn, is about the same degree higher than the static modulus.

c. The pulse velocity modulus is not significantly different from the modulus of elasticity in flexure, with both being higher than the static compressive modulus by about 2×10^6 psi.

d. The dynamic (impact) modulus both in compression and tension (tensile splitting) compares very closely with the secant static modulus.

3.2.3 OTHER FACTORS AFFECTING YOUNG'S MODULUS OF ELASTICITY OF CONCRETE

3.2.3.1 Density. It has been observed by many investigators that the elastic modulus of lightweight aggregate concrete is considerably lower than the values of normal weight concrete of comparable compressive strength and that the modulus appears to be a function of the weight. It is known that all mineral aggregates have about the same absolute specific gravity. The difference in weight of various types of concrete is therefore primarily the result of voids in the concrete, whether they be due to purposely entrained air or due to the vesicles in lightweight aggregate. From these considerations, it was suspected that it might be possible to obtain a satisfactory approximation by expressing the value of the elastic modulus by an empirical relationship of the form

$$E_c = aw^{3/2} \sqrt{f'_c} \qquad (3.5)$$

where

E_c = static modulus of elasticity of the concrete, psi

a = suitable constant

w = air-dried weight of the concrete at time of test, pcf

f'_c = compressive strength of the concrete at time of test, psi

Pauw[12] analyzed test data on various aggregates reported by various investigators, and, using the method of least squares, he established the constant a in the equation to be 33. The test data plotted were quite scattered partly because the elastic modulus values reported by the investigators were evaluated by differing test procedures, some investigators reporting the initial tangent modulus and others a secant modulus at some given stress level. In some instances, the method used to determine the static modulus was not even defined. In a few instances, only the wet weight of the concrete was reported. These data were used by adjusting the weight by applying reduction factors based on experience with similar aggregates.

Shideler[8] found that the static modulus of elasticity of various lightweight aggregate concretes varied from 57 to 82 percent of the modulus of elasticity of sand-gravel concrete.

3.2.3.2 Strength. Shideler[8] made an extensive series of laboratory tests on lightweight aggregate concrete and found that the modulus may vary widely for a given strength, depending on the mixture proportions and age of specimens. The modulus appears to increase more rapidly than strength at later ages.

It has been stated that, in general, an increase in strength is associated with an increase in the modulus. However, Troxwell and Davis[13] cautioned that

> this should not be construed to imply any definite relationship of these two characteristics, even though the ACI [American Concrete Institute] Code for reinforced-concrete design states that the modulus shall be taken as 1,000 times the compressive strength at age 28 days. Actual tests show that even when dealing with different combinations of the same materials, no universal relationship exists between strength and the modulus of elasticity, and when such variables as different aggregates are introduced, it is possible to produce concretes having the same modulus but having strengths differing by more than 100 per cent.

3.2.3.3 Aggregates. Higginson et al.[14] reported research to the effect that the modulus of concrete is to some extent dependent on the modulus of the aggregate (i.e., higher modulus aggregates produce

higher modulus cements); however, for a given cement paste, the modulus of the aggregate has less effect on the modulus of the concrete than can be accounted for by the volumetric proportions of the aggregate.

Neville[15] stated that the shape and surface characteristics of the coarse aggregate particles affect the stress-strain relation through their influence on microcracking but did not quantify the extent of this effect.

McDonald,[16] reporting on tests conducted to investigate multiaxial creep in concrete, concluded that it is possible to proportion concrete mixtures containing widely varying aggregate moduli with subsequent variations in concrete moduli having similar compressive strengths. The test program included concrete strength, three aggregate types having a modulus ranging from 3.8 to 13.65 $\times 10^6$ psi, and two moisture conditions. Stress-strain curves were determined for specimens uniaxially loaded to 2400 psi. Both moist-cured and air-dried specimens were tested. The air-dried specimens exhibited slightly higher strains at maximum load than companion moist-cured specimens. The average moduli of elasticity of specimens of equivalent strength containing the high-, medium-, and low-modulus aggregate were 6.38, 5.66, and 3.08 $\times 10^6$ psi, respectively.

3.2.3.4 Temperature. Nasser and Lohtia[17] studied the effect of elevated temperature on the modulus of elasticity and presented the data shown in Table 3.3. The heating of concrete A was begun at an age of 1 day while that of B was begun at 14 days. Both were sealed in metal containers when heated. Nasser and Lohtia found that elasticity is independent of temperature from 35 to 200° F starting at about 28 days. However, at higher temperatures, the effect was significant, and after a long period of exposure to 450° F, the elasticity was reduced to below 50 percent of that of the 70° F specimens. Philleo[18] and Saemann and Washa[19] studied the effect of heat and reached essentially the same conclusions as Nasser and Lohtia. Harmathy and Berndt[20] studied the effect of temperature on lightweight concrete and found that there was little effect on modulus of elasticity up to about 400° F, but the modulus gradually decreased to below 50 percent when the temperature was about 400° F.

3.2.3.5 Rate of Loading. Houk, Paxton, and Houghton[21] found in testing small beams 73 to 116 days in age that, when the load application of 90 percent of failure was extended over several months instead of several minutes as in conventional rapid tests, unit strains more than doubled. The modulus, therefore, of the long-term strain tests would be less than half the conventional modulus. Popovics,[22] in reviewing the stress-strain relationship for concrete, reached the following conclusions:

a. The testing conditions, such as the rate of loading, the number of load repetitions, the magnitude of the repeated stresses, etc., influence greatly the stress-strain diagram of concrete.

b. The presently accepted explanation is that the stress-strain diagram of a concrete under short-time loading deviates gradually from the straight line mainly because of progressive propagation of internal cracking in the specimen. Through this process, the aggregate content of concrete influences considerably the curvature of the stress-strain curve.

c. Even the theory of internal crack propagation cannot do better than to provide a qualitative description of the stress-strain relationship. For numerical approximation, empirical formulas are presented. The limits of validity and degree of approximation of such formulas are restricted.

3.2.3.6 Freezing and Thawing. This factor is known to be perhaps the most detrimental of all to the modulus of concrete. Walker[23] did early studies of this and reported his findings in 1944. He studied the effect of freezing and thawing of concrete made with different aggregates and reported that the dynamic modulus of concrete after 50 cycles of freezing and thawing ranged from 14 to 89 percent of the original modulus, and after 100 cycles, from 7 to 89 percent, depending primarily on the aggregate in the concrete. The effect of freezing and thawing on the modulus of elasticity of the concrete was found to be related to the durability of the aggregate. The harder, more dense, and less porous aggregate withstood freezing and thawing and thus suffered less reduction in modulus than did the softer, more porous aggregate. The best way to minimize damage of concrete by

freezing and thawing is to introduce entrained air by use of an air-entraining agent. This is effective because the numerous, well-dispersed minute air voids provide reservoirs for the relief of pressure created in the concrete due to freezing and results in less damage to the aggregates and cementing matrix.

3.3 POISSON'S RATIO

When a load is applied to a concrete specimen, the specimen is deformed. The deformation depends, among other things, on the magnitude of the load, the rate at which it is applied, and the elapsed time after the load application until the strain observation is made. A concrete cylinder under compressive stress will initially decrease in volume because of densification of the concrete. However, at some point, this decrease in volume will become zero and thereafter an increase will take place caused by internal cracking. The point of change announces the development of cracking to the point that the concrete is no longer a continuous body. Experiments show that when a material is subjected to axial compressive stress within the elastic limit, it deforms not only axially (decreases) but also laterally (increases). Under tension, the axial dimension increases and the lateral decreases. Figure 3.4 illustrates the effect of tensile and compressive stresses on a solid body. Poisson's ratio, therefore, is the ratio of the absolute value of the strain in the lateral direction to the strain in the axial direction. The general formula is

$$\nu = \frac{\text{lateral strain}}{\text{axial strain}} \qquad (3.6)$$

3.3.1 TEST PROCEDURES FOR DETERMINING POISSON'S RATIO

<u>3.3.1.1 Static Poisson's Ratio.</u> Static determinations are made by adding a third yoke and dial gage to a compressometer so that a magnified lateral strain can be measured along with the axial strain. These strains may also be measured electrically by mounting strain

gages on the surface of a specimen in the axial and lateral directions. Procedures for determination of static Poisson's ratio on concrete cylinders are included in ASTM 469[2] and CRD-C 19.[1] The formula for calculating Poisson's ratio statistically is as follows:

$$\nu = (\varepsilon_{t2} - \varepsilon_{t1})/(\varepsilon_1 - 0.000050) \tag{3.7}$$

where

ε_{t2} = transverse strain at midheight of the specimen produced by a stress corresponding to 40 percent of the ultimate load

ε_{t1} = transverse strain at midheight of the specimen produced by a stress corresponding to a longitudinal strain of 50 μin./in.

ε_1 = longitudinal strain produced by the stress at 40 percent of the ultimate load

3.3.1.2 Dynamic Poisson's Ratio. There are at least two techniques by which Poisson's ratio may be determined with sonic testing equipment. The more commonly used one is shown in CRD-C 18[1] and involves computations based upon two types of vibration of a specimen. Poisson's ratio may be calculated from the relation

$$\nu = \frac{E}{2G} - 1 \tag{3.8}$$

where

G = modulus of rigidity

The modulus of elasticity is usually determined on the basis of the fundamental resonant frequency of the specimen either longitudinally or transversely, and the modulus of rigidity is determined by the fundamental resonance in torsion. Both ASTM C 215[2] and CRD-C 18[1] recommend that when E and G are dynamic values Poisson's ratio should be designated as dynamic Poisson's ratio.

A much less common but sometimes used method of computation of dynamic Poisson's ratio involves the velocity measurement of either the transverse or Rayleigh wave along with the compressional wave. Very limited use, however, has been made of this method.

3.3.2 COMPARISON OF POISSON'S RATIO DETERMINED DYNAMICALLY AND STATICALLY

Neville,[15] in researching Poisson's ratio, found that it varies between 0.11 and 0.21 (generally 0.15 to 0.20) when determined statically for both normal weight and lightweight concrete.

Anson and Newman[24] found that a dynamic determination of Poisson's ratio yields higher values, especially at early ages, averaging about 0.24 as compared with 0.16 to 0.17 determined statically at stresses below 40 percent of ultimate.

3.3.3 OTHER FACTORS AFFECTING POISSON'S RATIO OF CONCRETE

3.3.3.1 Effect of Rapidly Alternating Loads and High Stresses. At high stresses or under conditions of rapidly alternating loads, a different behavior is observed. Probst[25] has shown a systematic increase in the value of Poisson's ratio with *stress repetition*. Brandlzaeg[26] shows a marked increase in Poisson's ratio at very high stresses. When the ratio is below 0.50, there is a decrease in volume of the specimen as a compressive load is applied, but Brandlzaeg's work indicates that when the stress is above 80 percent of ultimate there is an increase in volume as additional stress is applied, apparently because of internal cracking.

3.3.3.2 Effect of Aggregate Content. Poisson's ratio decreases with an increase in the content of normal weight aggregate, as shown by Neville's work[15] presented in Figure 3.5. Anson[27] found that Poisson's ratio for neat cement paste is 0.25 and that influence of richness of the mix is more noticeable in mortar than in concrete.

Under biaxial stress, Poisson's ratio has been measured to be 0.20 in compression-compression, 0.18 in tension-tension, and between 0.18 and 0.20 for compression-tension according to Kupfer, Hilsdorf, and Rüsch.[28]

<u>3.3.3.3 Creep Poisson's Ratio.</u> Neville[15] found that the ratio for creep strains which may be called creep Poisson's ratio is approximately the same as the elastic Poisson's ratio when the sustained load is uniaxial. Under sustained multiaxial compression, creep Poisson's ratio is smaller, ranging between 0.09 and 0.17; the value becomes smaller as the lateral stresses become larger relative to the axial stress. This means that the volume of concrete decreases with the progress of creep.

<u>3.3.3.4 Effect of High Temperature.</u> It has been demonstrated by a number of investigators that high temperatures, especially above 400° F, have a detrimental effect on the modulus of elasticity of concrete. Since Poisson's ratio is also a measure of strain under stress, it is reasonable to assume that high temperature will also be detrimental although there is a lack of test data regarding this. Philleo[18] made this statement regarding research work to investigate the effect of high temperature on concrete:

> Since both flexural and torsional frequencies were determined, it was possible to compute Poisson's ratio. There was a general tendency for Poisson's ratio to decrease as the temperature rose, although the results were erratic. The calculation of Poisson's ratio is very sensitive to errors in determining the resonant frequencies. A 1 percent error may produce as much as a 20 percent error in Poisson's ratio.

Higginson,[29] in studying the effect of steam curing on concrete, found that the modulus of elasticity and Poisson's ratio were not affected by steam curing of specimens.

3.4 CONCRETE STRENGTH

Tests to determine strength are the most common tests made to evaluate the properties of concrete because the strength is directly related to the load-bearing capacity of the concrete. The strengths most commonly determined are the compressive, tensile, or shear strength; however, in reality, the quantity measured by any test is probably a combination of two or more of these strength parameters. For example, the so-called uniaxial "compressive" strength test is standardized and widely used; however, the concrete does not fail in true compression.

Instead, the failure is the result of shearing and tensile stresses induced in the concrete specimen by the applied compressive force. Actually, all concrete materials eventually fail by one or another of the modes of tension and shear.

For pavements, strength of the concrete is of particular interest since it generally dictates the required thickness. Bending of the pavement slab under wheel loads produces compressive, shear, and tensile stresses in the concrete. Studies have shown that of these the required pavement thickness is controlled by the tensile stresses and the tensile strength. Since the tensile stresses are produced due to bending of the slab, as opposed to a direct tensile pull, the flexural strength test has been selected as most representative of the slab bending stress. However, the flexural strength test is one of the more difficult of the concrete strength tests to perform. As a result, many agencies resort to a correlation between the compressive or tensile splitting strength (which are much easier to determine) and flexural strength, especially for construction control and evaluation purposes.

3.4.1 COMPRESSION TESTS

Three types of compression test specimens are normally used: cubes, cylinders, and prisms. For pavement work in the United States, the general practice is to use the standard cylinder test (ASTM C 39-72[2]). The cylinders are usually 6 in. in diameter by 12 in. long and are cast in molds of steel or cast iron or are cored from the constructed pavement. Standard cylinders are of a height twice the diameter, but other height-to-diameter ratios may be used if an appropriate correction factor such as that developed by Murdock and Kesler[30] is used. Proper preparation of the cylindrical specimen for test is important. The ends of cast specimens may need grinding or other preparation and the ends of cored specimens must be capped or otherwise prepared to assure that the ends are plane and parallel and are normal to the axis of the specimen. Misalignments of as little as 1/4 in. in 12 in. have been shown to affect the compressive strength of the concrete.[31] Another important consideration in the conduct of the compression test is the

rate of loading which should be constant and between 20 and 50 psi per second. The compressive strength is expressed as P/A , where P is the maximum load in pounds and A is the cross-sectional area of the cylinder in square inches.

3.4.2 FLEXURE TESTS

The flexure test is the primary method for measuring the tensile strength of concrete for pavements because (a) it more nearly represents the stress induced by bending of the slab under wheel loading and (b) a satisfactory method for determining the tensile strength of concrete by direct tension has not been developed.

In the flexural tests of concrete, there are two methods to calculate the modulus of rupture; i.e., simple beam with third-point loading (CRD-C 16-66[1]/ASTM C 78[2]) and simple beam with center-point loading (CRD-C 17-69[1]/ASTM C 293[2]). The difference between the two methods is the loading arrangement. In the third-point loading method, the specimen is loaded by two point loads which are spaced so that the distance between the loads and the distances from the load to the nearest support of the specimen are equal. In the center-point loading method, the specimen is loaded by a single load located at the center of the specimen. The modulus of rupture is the maximum tensile stress at rupture computed from the flexural formula

$$R = \frac{Mc}{I} \tag{3.9}$$

where

R = modulus of rupture, psi

M = maximum bending moment at the section, in.-lb

c = distance from neutral axis to farthest fiber (one half the depth of the beam), in.

I = moment of inertia of the beam cross section, $in.^4$

Since the assumptions on which the flexure formula is based do not hold true at high stresses approaching failure, the modulus of rupture is thus a fictitious value; however, it is convenient for purposes of evaluation and is commonly used.

The modulus of rupture actually overestimates the tensile strength of the concrete and gives a higher value than would be obtained in a true tension test. This approximate value is based on the assumption in the calculation of the modulus of rupture that stress is linearly proportional to the distance from the neutral axis of the beam; the shape of the actual stress block under loads nearing failure is known <u>not</u> to be linear.

Because of the difference in loading arrangement of the two methods, expressions to calculate the modulus of rupture of concrete are different. The equations are given in the following paragraphs.

<u>3.4.2.1 Third-Point Loading.</u> If the fracture occurs within the middle third of the span length, the modulus of rupture shall be calculated as follows:

$$R = \frac{P\ell}{bd^2} \tag{3.10}$$

when

R = modulus of rupture, psi

P = maximum applied load indicated by the testing machine, lb

ℓ = span length, in.

b = average width of the specimen, in.

d = average depth of the specimen, in.

If the fracture occurs outside the middle third of the span length by not more than 5 percent of the span length, the modulus of rupture shall be calculated as follows:

$$R = \frac{3Pa}{bd^2} \tag{3.11}$$

where

a = distance between the line of fracture and the nearest support measured along the center line of the bottom surface of the beam, in.

As can be seen, if the fracture occurs at the load, the two equations are identical. If the fracture occurs outside of the middle third of

the span length by more than 5 percent of the span length, the results of the test should be discarded.

3.4.2.2 Center-Point Loading. The modulus of rupture is calculated as follows:

$$R = \frac{3P\ell}{2bd^2} \tag{3.12}$$

It can be readily seen that the modulus of rupture computed from the center-point loading is at least 1.5 times greater than that computed from the third-point loading. The difference increases when the fracture occurs outside of the middle third of the span length. The reason for the difference in the modulus of rupture between the two methods lies in the fact that the moment at the center of the span length in the center-point loading condition is 1.5 times greater than that in the third-point loading condition.

3.4.3 DIRECT TENSION TEST

There is no standard test for tension testing of concrete; however, for research purposes, a number of tension tests have been devised including testing of large briquets in tension and the use of glued or clamped plates on the ends of cylinders. Because of the difficulty of performing a direct tension test, there is some doubt as to the accuracy of the results obtained and the tensile strength of concrete is normally estimated based upon the results of the flexure test or by performing the splitting tension test. Kadlecek and Spetla[3] in a series of tests using the glued end plates concluded that the direct tensile strength of concrete can be reliably determined and the values are superior to those obtained by indirect methods and the use of conversion factors. The test can be used for both cast and cored cylinders or beams having a length-to-diameter or depth ratio of more than 2 without having to apply a correction factor because of influence from the end plates. Figure 3.6 presents an example of the relationships developed by Kadlecek and Spetla for the tensile strength of concrete determined by the flexure test, splitting tension test, and direct tension test and the compressive

strength using one concrete mix proportioning. Another fairly well recognized relationship[32] between the modulus of rupture from the flexure test and the direct tensile strength based upon the assumption that there is a linear change in stress from the neutral axis to the outer fiber of the beam is expressed as

$$T = \frac{R - 100}{1.4} \tag{3.13}$$

where

T = tensile strength, psi

A comparison of these two relationships indicates a difference, and it can be expected that the relationship will vary depending upon the test methods and the concrete mix proportionings used.

3.4.4 SPLITTING TENSION TEST

A different and slightly simpler method of measuring tension in concrete is the indirect tension test (ASTM C 496-69[2]). This method was developed coincidentally in Brazil and Japan in 1953 and is rapidly coming into general use. The test specimen is a conventional cylinder either cast or cored from the concrete in the same manner as for the compression test. The cylinder is loaded in compression along two axial lines through bearing strips of 1/8-in.-thick by 1-in.-wide hardwood plywood. The 1-in.-wide hardwood plywood strips distribute the compressive load over an area which is sufficiently narrow to avoid undue concentrations of stress. The strips compensate for surface irregularities to a certain degree; however, if the irregularities are excessive, capping along the axial lines may become necessary. The application of a compressive force along the two lines produces a triaxial stress distribution within the specimen, the horizontal stress being tension.[32] The cylinders fail suddenly along a vertical plane in the center region. The almost constant tensile stresses occur over approximately three quarters of the vertical plane between the two lines of load application. The magnitude of the average tensile stresses along this plane at the time of failure is considered to be the tensile strength computed as follows:

$$T = \frac{2P}{\pi \ell d} \tag{3.14}$$

where

T = splitting tensile strength, psi

P = maximum applied load, lb

ℓ = length of cylinder, in.

d = diameter of cylinder, in.

3.4.5 RING TENSILE STRENGTH TEST

The determination of the ring tensile strength of concrete has been approached using two methods; however, neither has been employed to any degree in pavement work. The first method uses thin discs with concentric holes in a diametrical compression test identical to the splitting tensile test. The basic idea is to change, by addition of the hole in the disc, the rather uniform tensile stress field which occurs across approximately the center three fourths of the failure plane such that the tensile stress component at the edge of the hole is increased. This insures that the origin of the fracture is known, and since the only component of stress acting at the edge of the hole is the unidirectional tensile stress, it represents the tensile strength of the concrete.

The second method utilizes the same type specimen and a uniform hydrostatic pressure is applied radially against the periphery of the ring. This pressure produces tangential tensile stresses and radial compressive stresses throughout the entire volume of the ring with a uniformly distributed maximum tensile stress occurring along the entire internal periphery of the ring. The radial compressive stress is quite small, so when failure occurs, it is the result of the tensile stress. These tests are fully described and the results of evaluation of the test for determining the tensile strength of mortars are presented in a report by Hoff.[33]

3.4.6 CORRELATION OF STRENGTH TEST

Several researchers have developed relationships between the

various strengths of concrete. In a summary of concrete strength relationships pertinent to pavements, Hammitt[34] performed a consolidated simple regression analysis of available reported data to develop the following relationships for strength expressed in pounds per square inch:

a. Compressive strength = -2123 + 10.02 flexural strength

b. Compressive strength = -1275 + 9.75 splitting tensile strength

c. Compressive strength = -1578 + 7.37 longitudinal shear strength

d. Flexural strength = 210.5 + 1.02 splitting tensile strength

Narrow and Ulbert[35] conducted a comprehensive study involving two aggregate types, cement contents ranging from 423 to 658 lb/cu yd, and ages of 7, 14, 28, and 90 days to develop a relationship between flexural strength and splitting tensile strength. They found that the ratio between flexural and splitting tensile strength varied immensely with the strength of the concrete as shown in Figure 3.7. Tables 3.4 and 3.5 and Figures 3.8 and 3.9 show the relationships between various strength tests of concrete.

Saucier[36] reported in a correlation of hardened concrete test methods and results in which he compared the results of unconfined compression, direct transverse shear (single and double plane) direct longitudinal shear, tensile splitting, and triaxial shear tests on cylindrical specimens using two types of aggregate and two cement contents. The results are shown in tabular form in his report and are not reproduced herein. Two conclusions reached as a part of Saucier's study[36] of importance to pavement are: "(a) the shear strength as determined from Mohr's theory more closely approximates the true shear strength of concrete than does that determined from direct tests, and (b) the physical properties of concrete are affected by the moisture condition of the test specimens at the time of test; where the stress is compressive, moist specimens yield lower indicated strengths."

3.4.7 FACTORS AFFECTING FLEXURAL STRENGTH

There are numerous factors which affect the flexural strength of concrete. Many of these are considered in selecting and proportioning

of ingredients. These include such factors as the amount of cement, ratio of the amount of water to the amount of cement, etc. In terms of characterizing the flexural strength for a particular mixture, additional factors are important.

3.4.7.1 Effect of Age. The increase in flexural strength of concrete with age is the result of chemical reactions of hydration occurring for an extended period of time. An illustration of time effects is shown in Figure 3.10.[37] To account for time effects, particular ages have been selected for determining flexural strength. Corps of Engineers and Portland Cement Association (PCA) design procedures are based on flexural strength determined at 90 days age, while FAA procedures are based on 28-day strength. Packard[38] states that the 90-day strength is from 110 to 114 percent of the 28-day strength.

3.4.7.2 Effect of Beam Size and Type Loading. The effect of beam size and type loading (center-point or third-point) is illustrated in Figure 3.11. The argument presented by Neville[4] to account for the differences is the so-called "weak link theory" in which it is postulated that concrete is not uniform and that the strength of the material within a beam varies. The larger the beam the higher the probability that a weak element will be exposed to a critical stress. Therefore, larger beams indicate lower strength than do smaller beams. Lindner and Sprague[39] also found that the flexural strength decreased as the size (depth) of the beam increased and attributed the decrease in strength to a differential shift in the neutral axis with change in the depth of the beam. The difference between the loading conditions may be explained by considering the moment distribution. For the center-point loading, the moment distribution is triangular with the maximum moment (stress) occurring at the beam center line, while for the third-point loading the distribution is rectangular between the loading points so that the maximum moment (stress) occurs over this entire length. The probability that a weak element will be exposed to a critical stress is larger for third-point loading than for center-point loading. Thus, third-point loading indicates lower strength than center-point loading. FAA, Corps of Engineers, and PCA design procedures are all

based on flexural strength determined from 6- by 6-in. beams loaded at the third points.

3.4.7.3 Effect of Curing Conditions. The temperature and moisture conditions (prevention of loss of moisture) during the curing period will significantly affect the strength of the concrete. Prevention of loss of moisture is necessary to insure that the chemical reactions of the hydration process occur. The effects of temperature vary with time. Temperatures higher than about 70° F will result in higher strength for a few days but lower strength at later periods. The lower bound is not definitive but is probably between about 40 and 50° F. Temperatures below about 70° F, but above an indefinite lower bound, will result in lower strengths for the first few days but will increase to normal levels with time. At temperatures below a certain minimum the strength increases with time but remains below what would have been obtained at about 70° F. At temperatures below freezing there is very little strength increase with time. ASTM C 31[2] and CRD-C 11[1] outline standard procedures for curing concrete beams to insure that results from tests of beams provide meaningful results. The applicability of the results to the characterization of the in-place pavement will, however, depend on the proper curing of the in-place concrete.

3.4.7.4 Effect of Aggregate. Singh[40] found that as the specific surface of aggregate is increased while maintaining other mixture proportions constant, the flexural and compressive strengths decrease. Walker and Bloem[41] found this same trend to be true as the maximum coarse aggregate size was increased.

3.4.7.5 Effect of Fatigue. Concrete in pavements is subject to the effects of fatigue. A fatigue failure occurs when the concrete ruptures under continuous repetition of loads that cause stress ratios (flexural stress to flexural strength) of less than unity. Flexural fatigue research[38] has shown that as stress ratios decrease the number of stress repetitions to failure increases. It has also been shown that:

a. When the stress ratio is not more than about 0.55, concrete will withstand virtually unlimited stress repetitions without loss in load-carrying capacity. Hence, concrete has a flexural fatigue endurance limit at a stress ratio of approximately 0.55.

b. Repetitions of loads with stress ratios below the endurance limit increase the ability of the concrete to carry loads with stress ratios above the endurance limit; i.e., the concrete fatigue resistance is improved.

c. Rest periods also increase the flexural fatigue resistance of concrete.

Fatigue effects are reflected in airfield pavement design procedures by selection of a conservative safety factor, or, where specific knowledge of loads and volumes is available, a more detailed analysis of fatigue effect is made.

3.4.8 UNIFORMITY OF FLEXURAL STRENGTH TESTS

A rather comprehensive study was conducted by the Ohio River Division Laboratories[42,43] to determine the variation in flexural strength results that could be expected both in the field and in the laboratory. Conclusions of this report are:

a. Considerable variation in flexural strength test results may be expected for specimens taken from the same concrete mix, even under the best controlled conditions of preparing, curing, and testing of the specimens.

b. A variation in flexural strength of 10 percent from the average values is not considered to be a satisfactory criterion for rejection of individual test results.

c. A variation in flexural strength of 15 percent from the average value may be considered significant where the difference cannot be attributed to variations in the concrete mix.

d. The rejection of individual values under any of the criteria considered has only a minor effect on the average strength of a group of specimens, and the effect becomes less as the number of specimens in the group increases.

3.5 FATIGUE OF CONCRETE AND ITS RELATION TO PAVEMENT PERFORMANCE

3.5.1 FATIGUE OF PLAIN CONCRETE

Like all other pavement materials, portland cement concrete is subjected to repetitive loads which are generally not considered in design. Although the flexural (tensile) stress induced by the traffic load is generally smaller than the ultimate static failure strength, excessive cracking may result and in some cases failure may occur.

Fatigue of concrete is described by Kesler[44] as a process of progressive, permanent internal structural change in a material subjected to fluctuating stresses and strains. The internal changes are generally damaging and may culminate in cracks or complete fracture after a sufficient number of fluctuations. The fluctuations in stresses and strains may occur as the result of repeated loads, temperature changes, and, perhaps, moisture content changes.

A number of technical papers have been published in recent years reviewing previous published works dealing with the fatigue of plain concrete. These appraisals are entered chronologically in the list of references.[44-50] In general, the fatigue property of plain concrete depends upon many factors, such as frequency of loading, sequence of load repetitions, rest period, stress gradient, and age of the concrete. Most of the investigations into the fatigue properties of plain concrete can be divided into two main types of tests: axial loading on cylindrical or prismatic specimens, usually under compressive loads, and flexural tests on simply supported beams.

There have in addition been a small number of indirect tensile splitting tests on cylinders tested on their sides, while some of the earlier work on road materials was done using cantilever specimens. Some tests conducted by the PCA were on long slabs continuously supported as in a road pavement slab. However, the majority of the work reported in the literature was done either in compression or in simple bending under repeated loading.

Fatigue tests on concrete beams were conducted as early as 1920 by Clemmer of the Illinois Department of Highways and were aimed at

producing design criteria for concrete pavement slabs.[51] The tests were conducted on sets of cantilever beams arranged radially like the spokes of a wheel and subjected in turn to repeated load by means of a truck wheel traveling around a circular track. On the basis of these tests, a design value of fatigue strength of 53 percent of the static ultimate strength was adopted for many years. From 1923 to 1928, Hatt and Crepps of Purdue University investigated the fatigue behavior of concrete cantilever beams subjected to completely reversed loading.[52-54] The tests suggested the existence of an actual fatigue limit of approximately 55 percent of the static ultimate strength.

The work that has been done on the fatigue of concrete in recent years has been reviewed for this study, and material properties will be presented in the following order:

- a. Endurance limit and fatigue strength.
- b. Age.
- c. Range of loading.
- d. Maximum strain.
- e. Load history.
- f. Frequency of loading.
- g. Rest period.
- h. Stress-strain relations.
- i. Shakedown limit.
- j. Effect of type of mix on fatigue properties.
- k. Crack propagation and fracture.

3.5.1.1 Endurance Limit and Fatigue Strength. Fatigue results are usually presented in the form of an S-N curve (stress versus log of the number of repetitions of load) as illustrated in Figure 3.12. If there is a break in the curve and it becomes asymptotic to a line parallel to the horizontal axis, the bounding stress is called an endurance limit or fatigue limit. Most metals and soils have an endurance limit but concrete probably does not, at least up to 10 million repetitions. Thus, the curve continues to slope downward as shown. The fatigue strength is the strength for any predetermined number of

repetitions of load, usually the end point of the curve. Kesler[44] has stated that a typical fatigue strength at 10 million repetitions is about 55 percent (applied stress to static strength ratio). Hence, the commonly used endurance limit of 50 percent is perhaps justified in practice as a design value.

Fatigue strength data can be utilized effectively only if it has been examined statistically. McCall[55] added this important dimension to the presentation of fatigue data as shown in Figure 3.13. The usual fatigue curve is that shown for a probability of failure P of 0.5. Consideration of the curves in Figure 3.13 leads to the conclusion that safer and more efficient designs should be based on the curves with lower probabilities.

Raju[56] found that the fatigue strengths of paste, mortar, and concrete are about the same when expressed as a percentage of static ultimate strength. Many variables such as cement content, water-cement ratio, curing, entrained air, aggregate, etc., that affect static ultimate strength affect fatigue strength in a similar proportionate manner.

3.5.1.2 Age. Linger and Gillespie[57] have reported that fatigue strength increases with age, especially during the first 3 months. However, in their research they did not use the static ultimate strength at the time the fatigue tests were run but used rather the 28-day strength as the basis for comparison. Kesler[44] suggested that since the static strength also increases with age, the fatigue strength relationship could be independent of time, if the static strength of the concrete was used at the same age of curing that the fatigue tests were conducted. However, in an earlier publication, Kesler and Siess[58] stated that specimens for fatigue testing should be at least 3 months old before testing.

3.5.1.3 Range of Loading. For fatigue in both flexure and compression, the upper limit of the stress (fatigue strength) is increased substantially as the range of stress is decreased. The value of 55 percent of the static ultimate strength for the fatigue strength of concrete referred to earlier is based on specimens repeatedly loaded to a maximum

in either tension, compression, or flexure. Murdock and Kesler[59] conducted a series of fatigue tests to determine the effect of range of stress on flexural fatigue behavior. Endurance curves were produced for several values of minimum/maximum nominal stress. Some results are shown in Figure 3.14. It can be seen that if the range of loading is changed the maximum repeated load that can be sustained for a given life is also changed. In other words, the fatigue behavior is dependent upon the type of stress as well as the stress gradient. Ople and Hulsbos[60] studied the behavior of concrete prisms with a compressive stress gradient. In these tests, the specimens were loaded eccentrically to produce the required stress gradient across the section. This gradient was shown to have a marked effect on fatigue life. For specimens having the same peak compressive stress, an increase in fatigue strength of about 17 percent of static ultimate strength was achieved on specimens having a zero to maximum strain distribution, when compared with uniformly loaded specimens (conditions of zero stress gradient).

3.5.1.4 Maximum Strain. Similar to asphaltic concrete for flexible pavements, concrete pavement can be failed in fatigue by a repeated load of lower magnitude than the static ultimate load. However, the gross strains at failure in a specimen subjected to repeated load are larger than in a specimen loaded statically. Hilsdorf and Kesler[61] found that the maximum tensile strain of concrete subjected to repeated flexural loads appears to be independent of fatigue life.

3.5.1.5 Load History. In his review of research work published prior to 1959, Nordby[45] concluded that concrete possesses a property similar to strain hardening in metals. Loading repetitively at less than the fatigue strength resulted in raising the fatigue strength and/or stiffening of the specimens. Clemmer[51] in 1922 indicated that such understressing increased the static strength approximately 5 percent, while LeCamus[62] in 1946 found that the static strength increased as much as 8 to 15 percent. Bennett and Muir[63] in 1967 found that for some high-strength concretes the static strength was increased approximately 11 percent after the concrete had been subjected to several million

repetitions of load between 0.5 and 0.7 of the static ultimate strength. Bate[64] in 1956 noted that beams subjected to a similar stress history resisted a greater number of load repetitions than would normally be expected when tested later at a higher stress level.

In 1966, Hilsdorf and Kesler[61] published a technical paper in which the effect of random loading on the fatigue behavior of concrete was reported. The tests were conducted using varying flexural stresses. Three different loading programs were used:

a. After a number of repetitions of load at a given stress, the stress was increased to a new level and maintained until failure.

b. After a number of repetitions of load at a given high stress, the stress was decreased to a lower level and maintained until failure occurred.

c. Two stress levels were continuously alternated until failure, the lower being first in the sequence.

The high levels of stress in the three loading programs were of different magnitudes and were below the static ultimate strength of the concrete.

Hilsdorf and Kesler[61] drew the following conclusions based on their results:

a. The fatigue strength and life of concrete subjected to repeated loads of varying magnitude are influenced by the sequence in which these loads are applied. Consider a test in which the maximum stress level is changed only once. Then the fatigue life of a specimen is larger if the higher stress level has been applied first compared to the fatigue life of a specimen in which the lower stress level was applied first. A relatively low number of repetitions of high loads can in fact increase the fatigue strength of concrete under a lower load beyond the fatigue strength of concrete which has not been previously loaded.

b. If the upper stress level in a fatigue test is varied between two values continuously during the test, the lower value being constant, the fatigue life decreases with increasing magnitude of the higher stress level and also with increasing number of repetitions under the higher stress. Loads corresponding to stress levels less than that which would normally cause failure, can contribute to the damage initiated by previously applied higher loads.

3.5.1.6 Frequency of Repeated Loading. Since the static strength of concrete depends critically on the rate of loading,[58] it might be

expected that fatigue performance would also be affected by the speed of testing. Limited investigations indicate, however, that the effect is not great.

Kesler[65] and Assimacopoulos, Warner, and Ekberg[66] conducted flexural tests on concrete at different rates of loading and found that frequency of loading, between 70 and 900 repetitions per minute, has no significant effect on fatigue strength. However, frequencies as low as 10 repetitions per minute may result in slightly lower fatigue lives. This behavior is similar to reported fatigue results with asphaltic concrete.

3.5.1.7 Rest Period. Although the frequency of loading has no significant effect on fatigue strength of concrete, it was found that periodic rest periods increase the fatigue life of the concrete specimens. Hilsdorf and Kesler[61] conducted a series of tests with rest periods of different lengths; i.e., of 1, 5, 10, 20, and 27 minutes applied after each 10-minute loading period (i.e., approximately every 4500 repetitions). A sixth group examined the effect of a single rest period of greater magnitude. No difference was found in the effect of 5-minute or longer rest periods, but 1-minute rest periods had a less beneficial effect, as shown in Figure 3.15. Five-minute rest periods increased the fatigue strength from 62 to 68 percent of the static strength. The fatigue strength when there were no rest periods was 62 percent because the minimum load was an appreciable percentage of the maximum load.

Hilsdorf and Kesler[61] explained that during periods of rest, the specimens were subjected to a constant load substantially less than the maximum repeated load. The recovery of each specimen was qualitatively identical to that observed in relaxation studies; i.e., the initial rate of recovery is great, but rapidly diminishes. Consequently, it appears that no additional benefits are derived when rest periods extend beyond 5 minutes.

3.5.1.8 Stress-Strain Relations. Similar to the behavior of asphaltic concrete, the elastic modulus of portland cement concrete subjected to repeated loads generally decreases with load repetitions.

Linger and Gillespie[57] studied the mechanics of concrete fatigue and fractures and found that the secant modulus of elasticity of concrete reduced as the number of repetitions increased. Such a relationship is shown in Figure 3.16, in which the maximum repeated stress and strains were used. It can be seen that the magnitude of the modulus computed from the maximum stress and strains drops rapidly during the early life of a fatigue specimen. It then decreases gradually until a short time before failure when there is a further rapid drop in magnitude. This is significant in practical design work because the modulus computed at failure may be only about 60 percent of its maximum or original value.

Hilsdorf and Kesler,[67] in their work on cumulative damage on beam specimens, concluded that fatigue behavior could be explained in terms of accumulated strain based on the following assumptions:

a. Failure occurs as soon as the total surface strain reaches a limiting value of 250 μin./in.

b. After n_1 repetitions of load under an alternating stress s_1, further strain increase due to repeated loading under an alternating stress s_2 will follow a curve identical with the strain function of s_2 but displaced along the repetition ratio (n/N) axis.

c. Repeated loading at first produces an increase in true static strength due to relief of shrinkage stresses followed by a decrease as cracks develop.

A comparison between stress-strain relationships, natural frequency, and number of load repetitions was made by Gatfield of the Road Research Laboratory[68] on rectangular-section beams supported at the nodal points and vibrated in the fundamental mode of free vibration. These tests indicated marked changes in both vibration characteristics and stress relationships as cracks developed under repeated loading. Cracks became visible to the naked eye when the frequency had dropped to about 70 percent of its original value. Commenting on Gatfield's paper, Welch[69] concluded that there was a critical value of surface strain which would lead to fatigue of the beam and that this strain could be related directly to the 5 percent deviation point of the static stress-strain curve of standard flexural beam strength test specimens, the critical

strain value increasing with the flexural strength of the concrete. He also concluded that the critical dynamic strains for lean concrete were substantially lower than for structural concrete.

3.5.1.9 Shakedown Limit. In the service lifetime of a portland cement concrete pavement, the possibility exists that an excessively heavy traffic load may induce stresses in the pavement of a magnitude very near to the ultimate strength of the concrete.

Shah and Winter[70] attempted to determine whether the internal damage caused by a few repetitions of near ultimate loading would weaken concrete for subsequent loading. Prismatic specimens were loaded up to 20 repetitions with stresses of the magnitude of 83 to 100 percent of the short-time ultimate strength. They reported that the shakedown limit for concrete was about 90 percent of the ultimate static strength. Repeated loads below the shakedown limit do not change the load-carrying capacity of concrete as determined by a single loading, but loads above this limit damage the integrity of the concrete. The shakedown limit appears to be near the critical load at which the volume of concrete under compressive loading ceases to decrease and the microcracks through the mortar sharply increase.

3.5.1.10 Effect of Type of Mix on Fatigue Properties. A few authors mentioned the effects of the type of mix (i.e., mix proportions, type of aggregate, curing conditions, and moisture content) on fatigue performance, but most gave few details. Although in most cases the applied stresses were given as a proportion of the static ultimate strength, it was not always clear which particular strength value was used as a reference.

In Nordby's review[45] in 1958, it was stated that fatigue strength (expressed as a proportion of static ultimate strength) decreased slightly with leaner mixes and higher water-cement ratios, but the data at that time were very limited.

In 1959, Antrim and McLaughlin[71] compared the fatigue strength in compression of air-entrained concrete with normal concrete and concluded

that (again related to static ultimate strength) there was no significant difference. This work was later extended to include lightweight aggregates[72] of low strength (3800-psi compression) and high strength (6000 psi). There was no significant difference in fatigue strength (expressed as a proportion of the static strength) between either of the two lightweight mixes or between them and the "normal" concrete of the 1959 investigation. An extrapolated value of about 55 percent of static ultimate strength at 10^7 repetitions under repeated loading was indicated.

3.5.1.11 Crack Propagation and Fracture. Studies have been reported on the behavior of cracks in plain concrete during fatigue testing. In most cases, small cracks have been observed to open and close during repeated loading with propagating, indicating that microcracks exist in the concrete long before failure becomes imminent.[73,74] At Lehigh University,[66] it was observed that failure tended to occur at the matrix-aggregate interface rather than through the aggregate, at any rate for small aggregates (3/8-in. sandstone and quartzite gravel).

Kaplan[75,76] suggested that under static loading the initiation of microcracking probably marked the beginning of the failure process. Initial fracture may result from the formation of multiple cracks in the cement mortar, with the aggregate forming a crack barrier. It is reasonable to suppose that a similar mechanism exists for the propagation of fatigue cracks.

Studies of the applicability of fracture mechanics to the failure of concrete have increased in recent years. Much of this work has been directed toward the behavior of the material during static failure but some work has also been done in relation to fatigue behavior. Glucklich[77] made a study of the influence of microcracking on all of the mechanical properties, including creep and fatigue, and gave some information on the propagation of fatigue cracks through mortar beams tested in flexure. He found that the critical crack length to cause

failure under fatigue loading agreed closely with that corresponding to static stress conditions, on the basis of the critical strain energy release rate. Kaplan[76] discussed the relationship of strain energy release rates to fracture mechanism of concrete with particular reference to the absorption of energy by multiple cracking and by the effect of aggregate particles forming crack barriers. Similar observations were made by Jones[78] in considering the probable mechanism of failure of concrete under quasistatic loading. It appears likely that the mechanism of fatigue failure will be very similar and can be characterized by an initial breakdown of the bond between the cement matrix and the aggregate, followed by progression of cracks across the mortar. A propagating fatigue crack will be arrested from time to time when it reaches a stone, and then the above process will be repeated until finally the rate of release of strain energy is sufficient to overcome the remaining cohesive forces and complete fracture occurs.

3.5.2 PAVEMENT DESIGN CONSIDERING FATIGUE PROPERTY OF CONCRETE

In the discussions presented in Section 3.5.1.5, it was concluded that the sequence of stress level affects the fatigue life of a concrete specimen. When the results of the first two programs of Hilsdorf and Kesler's[61] work were compared with the commonly used Miner hypothesis[79] which assumes linear accumulation of damage, it was found that when the lower stress level was applied first the Miner theory was unsafe, while where the higher stress level was applied first, the Miner theory was conservative. The results of the third program, where load was repeatedly varied, showed that the fatigue strength decreased as the ratio of the number of repetitions at the high stress level to those at the low stress level increased. The fatigue strength also decreased for a given maximum stress level as the difference between the two stress levels was increased.

Kaplan[75,76] concluded that because Miner's hypothesis is easy to apply it is frequently used, even though it does not give accurate predictions of failure of concrete. Consequently, design curves,[61] as

shown in Figure 3.17, incorporating the probability of failure, have been developed so that when Miner's hypothesis is applied to them reasonable results are obtained.

Of the current design manuals for portland cement concrete pavements, the PCA airfield design manual[38] is the only one which has an alternate analysis by which cumulative fatigue analysis may be used with the entire mix of aircraft anticipated at the airport.[80] The concept utilizes a typical fatigue curve for concrete pavements. The procedure is described in detail by Witczak[81] in his state-of-the-art report on pavement performance models and is therefore not repeated in this report. Also, in Chapter 3 of this report, the endurance limit, stochastic nature of fatigue, and fatigue failure concept of concrete pavements were discussed in detail; readers are urged to consult these chapters.

3.6 MISCELLANEOUS PROPERTIES AFFECTING PAVEMENT PERFORMANCE

There are many factors that affect pavement performance, such as reinforcement, joints, foundation, etc., that are not specifically part of the concrete and therefore are not considered in this report. However, there are, in addition to deformation characteristics (E and ν) and strength characteristics, other concrete properties which affect pavement performance. Some of these are discussed in the following paragraphs.

3.6.1 DURABILITY

With respect to durability of concrete, perhaps the first item that should be considered is resistance to freezing and thawing. All concrete contains void space capable of containing water that can freeze and exert pressure. This problem has largely been eliminated, however, by the use of air-entraining agents in the mortar matrix of the concrete which entrain minute air bubbles that, in turn, provide space large enough to accommodate the increase in volume created by the freezing water.

3.6.2 CEMENT-AGGREGATE REACTION

Another problem related to the durability of pavements is reactive aggregate. Virtually all aggregate is reactive with cement to some degree, and sometimes a small amount is beneficial in increasing the aggregate bond to the paste. But a highly reactive aggregate will destroy the concrete. If aggregate with a good service record is employed, there is no problem, but if marginal aggregate must be used, a low-alkali cement should also be used. If sulfate attack is possible, a sulfate-resistant cement should be used.

3.6.3 SURFACE ABRASION

Abrasion is defined as the wearing away of a concrete surface by a friction process. For airfield pavements, there is abrasive action from the wheels of vehicles as well as heat from jet engines and abrasive particles blown onto the concrete surfaces by jet exhausts and propeller backwash. It has been demonstrated that the cement factor, water-cement ratio, air content, and curing of concrete are all important factors. Probably, strength and curing are the most important factors in controlling abrasion resistance. It is well established that the longer the concrete is kept moist after the set, the better the strength and abrasion resistance. Data indicate that abrasion resistance increases rapidly with strength up to a point, depending on aggregate and curing conditions, but beyond that point, increase in strength has very little effect on abrasion resistance. This point may come between compressive strengths of 4000 and 6000 psi. Collins and Waters[82] data indicate that the initial rate of wear of 2000-psi concrete is about five times that of 4000-psi concrete. They also indicate that the type of aggregate used has an important effect on the later stages of wear of lower strength concrete but little effect on concrete above 6000 psi.

3.6.4 DEICER SCALING

Deicers, usually calcium chloride, applied to concrete surfaces to keep them free of snow and ice, give rise to a serious problem of

surface scaling. Entrained air in concrete has a very beneficial effect on reducing surfacing scaling. Other beneficial factors are high compressive strength (4000 to 6000 psi); low slump, clean, durable, well-graded aggregate; and sound consolidating, finishing, and curing practices.

3.6.5 VOLUME CHANGE

Shrinkage and volume change not related to detrimental chemical attack must be considered in airfield pavement construction. When volume changes are restrained by foundations, connecting members, or reinforcements, however, stresses are produced which may cause severe cracking. Plastic shrinkage in freshly mixed concrete is due to bleeding, absorption of water by aggregates, loss of water through improper curing, thermal change, and chemical hydration. Cracking can occur the first few hours after placement and occurs principally because of excessive rapid evaporation or loss of water. Pavement construction is especially susceptible to this. Corrective measures are all directed toward reducing the rate of evaporation or the total time evaporation can take place. Autogenous shrinkage (cement hydration) is a function of two opposing factors: (a) expansion of new gel due to absorption of free pore water and (b) shrinkage of gel due to extraction of water by reaction with the remaining unhydrated cement. The sum total is usually a slight amount of shrinkage (approximately 0.007 percent). Shrinkage increases with increased fineness of the cement and appears to be greater for low-heat cements. Thermal changes caused by variation in ambient temperatures are important in pavements since differential gradient at night may cause a contraction of the top surface relative to the bottom, thus tending to lift the slab ends above the subgrade, causing cracking under stress. The thermal coefficients of the aggregate seem to have the greatest effect. Siliceous aggregates have a higher coefficient (4.5 to 6.5 millionths per degree Fahrenheit) than limestone, basalt, granite, and gneiss (1.2 to 4.5), and thus are more damaging. The last type shrinkage to be considered is drying shrinkage. Washa[83] had the following to say on the combined effect of unfavorable factors on drying and shrinkage:

> It has been observed and experimentally shown that the cumulative effect of the individual factors that increase drying shrinkage can be very large, and that the combined effect is the product rather than the sum of the individual effects. Calculations have shown that the use of less favorable construction practices--concrete discharge temperature of 80 F rather than 60 F, a 6- to 7-in. rather than a 3- to 4-in. slump, a 3/4-in.-max aggregate size rather than 1-1/2 in., and too long a mixing and waiting period--could be expected to increase shrinkage 64 percent. If, in addition, a cement with high-shrinkage characteristics, dirty aggregates of poor inherent shrinkage quality, and admixtures that increase shrinkage are used the resultant shrinkage could be about five times as large as the shrinkage that would be obtained with the best choice of variables.

There are shrinkage-compensating cements presently on the market that offer promise for offsetting the detrimental effects of shrinkage, but there are problems involved in mixing, placing, and curing concretes containing these cements.

3.6.6 EFFECT OF HIGH TEMPERATURES ON CONCRETE

This effect was discussed earlier in this report in regard to strength and modulus of elasticity but should be considered here in light of the effects of jet blasts on the pavement surface. It has been demonstrated experimentally that temperatures below 400° F have little detrimental effect on concrete but have a much larger effect above 400° F. The presence of moisture in concrete is apparent in fire tests on concrete where excessive moisture at the time of fire is the primary cause of spalling. Concrete appears less affected where the concrete does not contain silica. Basic igneous rocks, crushed brick, and blast-furnace slag withstand fire damage better. Concrete that is heated to 450° C (840° F) or above will contain some free lime, the amount depending on the temperature and duration of heating, among other things. If limestone aggregates are present, they will decompose and yield free lime at temperatures of more than 600 to 700° C (1112 to 1292° F). Upon exposure to normal air after cooling, the free lime will hydrate and carbonate, usually causing surface scaling and possibly more deeply seated expansion and cracking.

3.6.7 DELETERIOUS SUBSTANCES

The effect of the individual aggregate particle on the durability of concrete depends primarily on its volume stability, provided it is not chemically reactive with the cement. A good aggregate must be able to resist excessively large or permanent changes in volume when subjected to potentially destructive agents such as freezing and thawing, heating and cooling, or wetting and drying. The volume change of unsound particles may cause deterioration of concrete ranging from localized pitting (popouts) and scaling to extensive cracking and deep-seated disintegration. The usual method for measuring soundness of aggregate is ASTM C 88[2] (sodium or magnesium sulfate soundness test).

3.7 OTHER MEASUREMENTS

3.7.1 RAPID MEASUREMENT OF WATER AND CEMENT CONTENT

The need for more rapid techniques to evaluate concrete quality has long been recognized. Much effort has been made recently to develop rapid (less than 15-minute) field methods for determining the water and cement contents of fresh concrete. In May 1975, a conference on rapid testing of fresh concrete was held in Champaign, Ill., sponsored by the U. S. Army Engineer Construction Engineering Research Laboratory (CERL). The purpose of the conference was to acquaint potential users with recent developments in field techniques. Eight papers were published in the conference.[84] Discussions were concentrated on the use of chemical, mechanical, and nuclear techniques to determine water and cement contents of fresh concrete. Data were also presented relating water and cement content test results to concrete strength potential.

3.7.2 SLUMP, FLOW, AND BALL TESTS

Consistency is a practical consideration in securing a workable concrete. Slump, flow, and ball tests were all designed to measure the consistency of fresh concrete. While these tests give only approximate

measures of this property, they do define ranges of consistency well enough for most practical work.

The slump test is made by measuring the subsidence of a pile of concrete formed in a mold which has the shape of a truncated cone. Owing to its simplicity, the test is adapted to field use and is widely used.

The flow test is made by jogging a pile of concrete on a metal table and noting the spread of the pile as a percentage of the original formed diameter. Although the results are more reproducible than those of the slump test, the flow test is largely limited to laboratory use because of the unwieldiness of the apparatus.

A ball test devised by Kelly appears to give promise of convenience and utility in determining the consistence of fresh concrete as it rests in a container such as a buggy. As the result of a large number of comparative tests made in the field and in the laboratory, good correlation was found between ball penetration and slump, with 1 in. of penetration equaling approximately 2 in. of slump.

3.7.3 AIR ENTRAINMENT MEASUREMENTS

The purpose of air entrainment in concrete is to make the concrete frost-resistant. For each mix, there is a minimum volume of voids required for protection from frost. It was found that this volume corresponds to 4 percent of the volume of mortar, and it is essential that the air be distributed throughout the cement paste.

Three methods have been used for determining the amount of entrained air in fresh concrete: (a) a volumetric method; (b) a gravimetric method; and (c) a pressure method.

In the volumetric method, the unit weight of a given mass is determined, after which the volume of the solids is determined by displacement in water in a pycnometer.

In the gravimetric method, the weight of a known sample is determined, from which is calculated the solid volume of the ingredients from their proportions and specific gravities. The amount of entrained air is taken as the difference between the observed volume of the sample

and the calculated solid volume. Considerable error, possibly greater than 1 percent, may arise because the specific gravities and actual proportions are not generally known with sufficient precision.

In the pressure method, the sample is placed in a special container called an "air meter" which has a transparent tube projecting from the cover. Water is added until the tube is nearly filled. A known pressure is then applied to the top of the tube, and the deflection of the water column is observed. The apparent reduction in volume of the mass of concrete and water is due to compression of the entrained air, the volume of which is then calculated through the application of Boyle's law. Some air meters are calibrated to read air content for a given applied pressure. The pressure method is considered to be the most dependable of the three and has found wide use.

Table 3.1

Young's Modulus of 6- by 6- by 30-In. Concrete Beams (after Philleo[6])

Number of Specimens	Average Static E* 10^{-6} psi	Ratio of Resonance E to Static E	Ratio of Pulse E to Static E
13	4.52	1.00	1.17
13	4.89	1.05	1.16
13	5.21	1.01	1.13
13	5.60	0.96	1.04
13	6.03	0.94	1.01

Note: Specimens tested at 28 days.
* Static E is secant modulus at 15 percent ultimate.

Table 3.2

Young's Modulus of 6- by 12-In. Concrete Cylinders (after Philleo[6])

Number of Specimens	Average Static E* 10^{-6} psi	Ratio of Resonance E to Static E	Ratio of Pulse E to Static E
9	3.92	1.21	1.54
9	4.45	1.11	1.42
9	4.78	1.01	1.31
9	5.08	1.02	1.27
8	5.50	0.92	1.10

Note: Specimens tested at 28 days.
* Static E is initial tangent modulus.

Table 3.3

Ratio of Modulus of Elasticity at Various Temperatures and Ages to That at 70° F for Two Concretes (after Nasser and Lohtia[17])

Concrete	Age days	Modulus of Elasticity E_c at 70° F, 10^6 psi	Ratio of E_c at Indicated Temperature to E_c at 70° F						
			35° F	160° F	250° F	300° F	350° F	400° F	450° F
A	14	5.05	0.81	0.84	0.80	0.77	0.58	0.51	0.44
	28	5.12	0.86	0.89	0.75	0.72	0.51	0.48	0.40
	91	5.20	0.96	0.94	0.68	0.64	0.51	0.42	0.35
	180	5.30	1.04	0.98	0.64	0.60	0.49	0.39	0.32
B	14	5.20	--	0.86	0.86	0.80	0.62	0.51	0.44
	28	5.25	--	0.90	0.86	0.76	0.58	0.46	0.39
	91	5.40	--	0.97	0.88	0.71	0.53	0.41	0.33
	180	5.50	--	1.02	0.88	0.67	0.50	0.37	0.32

Table 3.4

Variability of Results of Tests on Tensile Strength of Concrete (after Neville[15])

Type of Test	Mean Strength psi	Standard Deviation Within Batches psi	Coefficient of Variation percent
Splitting tensile	405	20	5
Direct tension	275	19	7
Modulus of rupture	605	36	6
Compression cube	5980	207	3-1/2

Table 3.5

Relation Between Compressive and Tensile Strengths of Concrete (after Neville[15])

Compressive Strength of Cylinders psi	Strength Ratio: Modulus of Rupture* to Compressive Strength	Strength Ratio: Direct Tensile Strength to Compressive Strength	Strength Ratio: Direct Tensile Strength to Modulus of Rupture*
1000	0.23	0.11	0.48
2000	0.19	0.10	0.53
3000	0.16	0.09	0.57
4000	0.15	0.09	0.59
5000	0.14	0.08	0.59
6000	0.13	0.08	0.60
7000	0.12	0.07	0.61
8000	0.12	0.07	0.62
9000	0.11	0.07	0.63

* Determined under third-point loading.

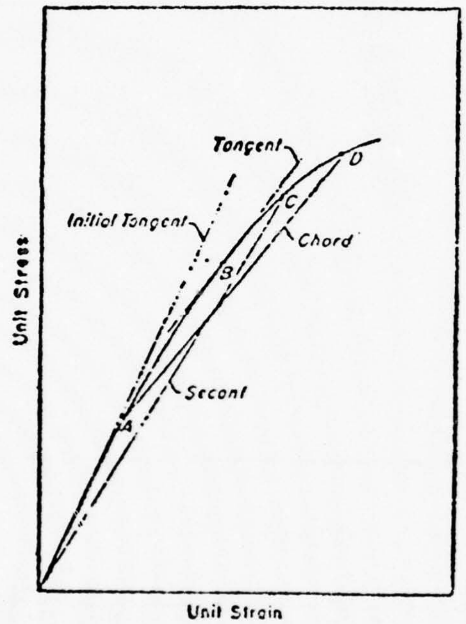

Figure 3.1. Various forms of static modulus of elasticity (compressive)

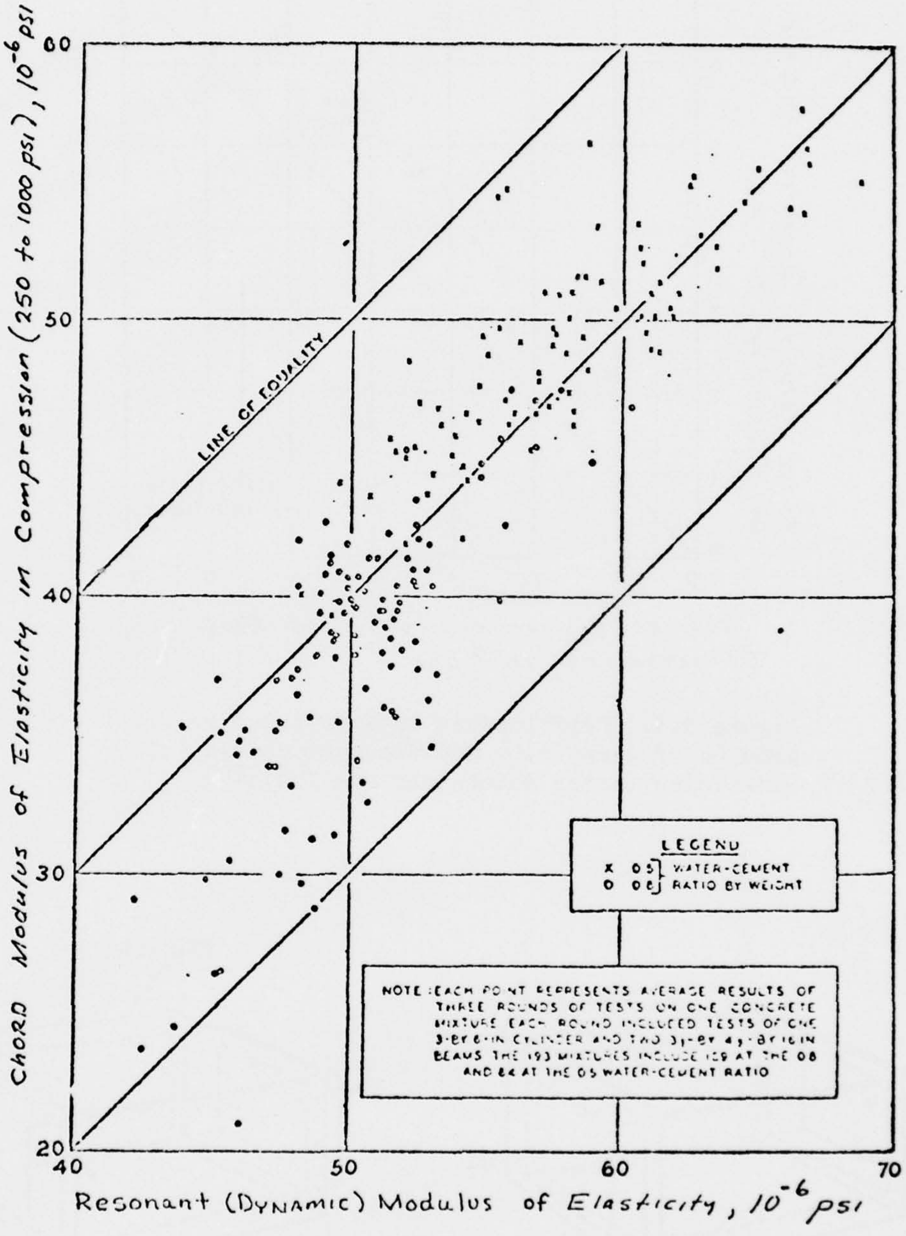

Figure 3.2. Relationship between static compressive modulus of elasticity and resonant modulus of elasticity (after Mather[7])

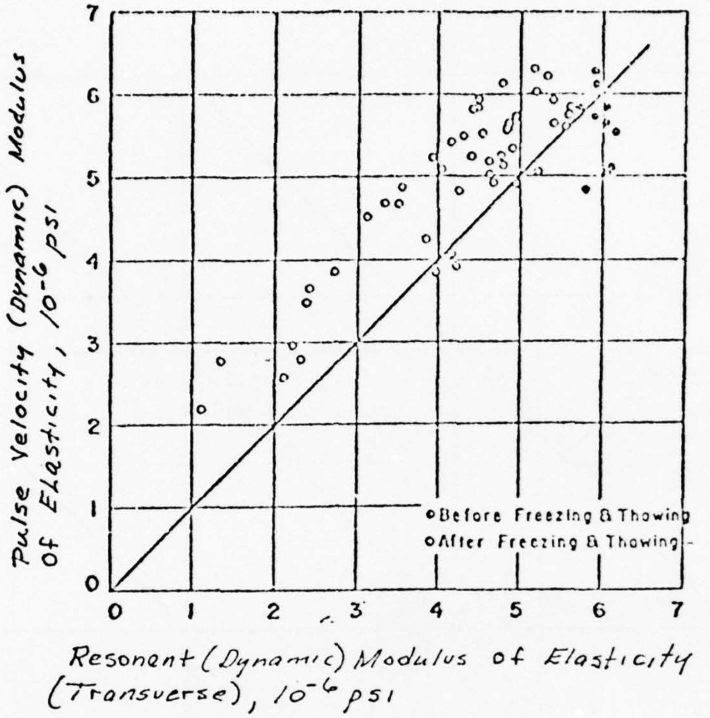

Figure 3.3. Relationship between pulse velocity modulus of elasticity and resonant modulus of elasticity (after Batchelder and Lewis[10])

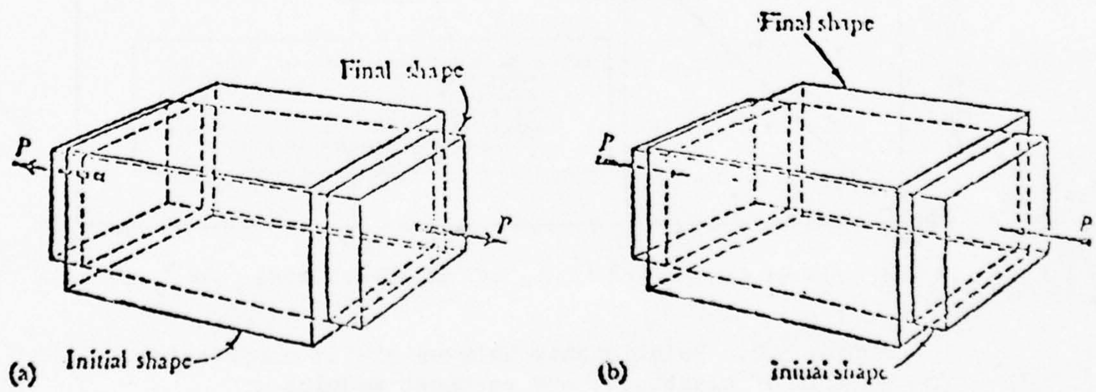

Figure 3.4. Lateral contraction and expansion of solid bodies subjected to axial forces (Poisson effect)

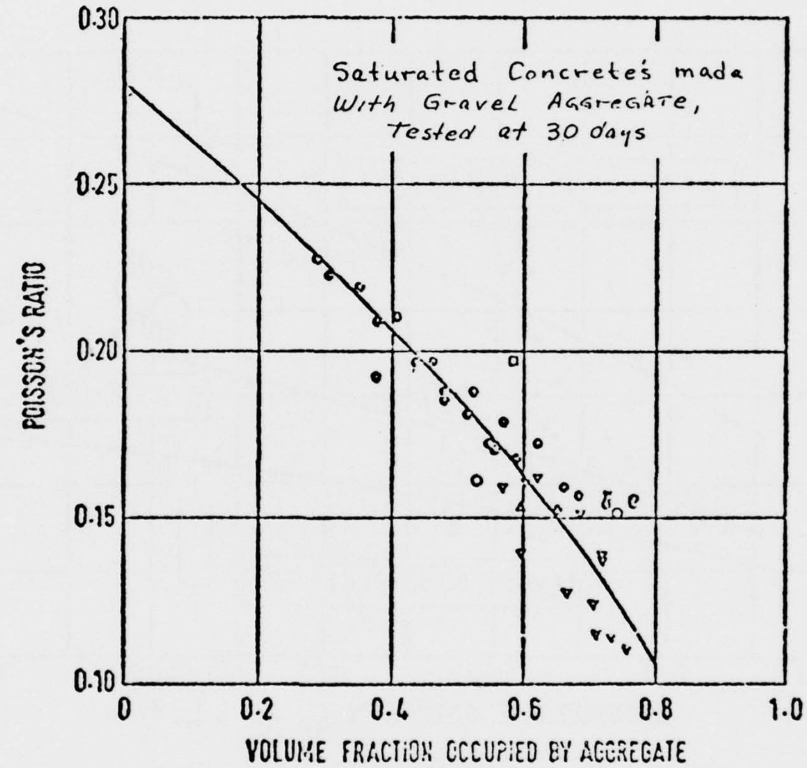

Figure 3.5. Relation between Poisson's ratio and volumetric content of aggregate (after Neville[15])

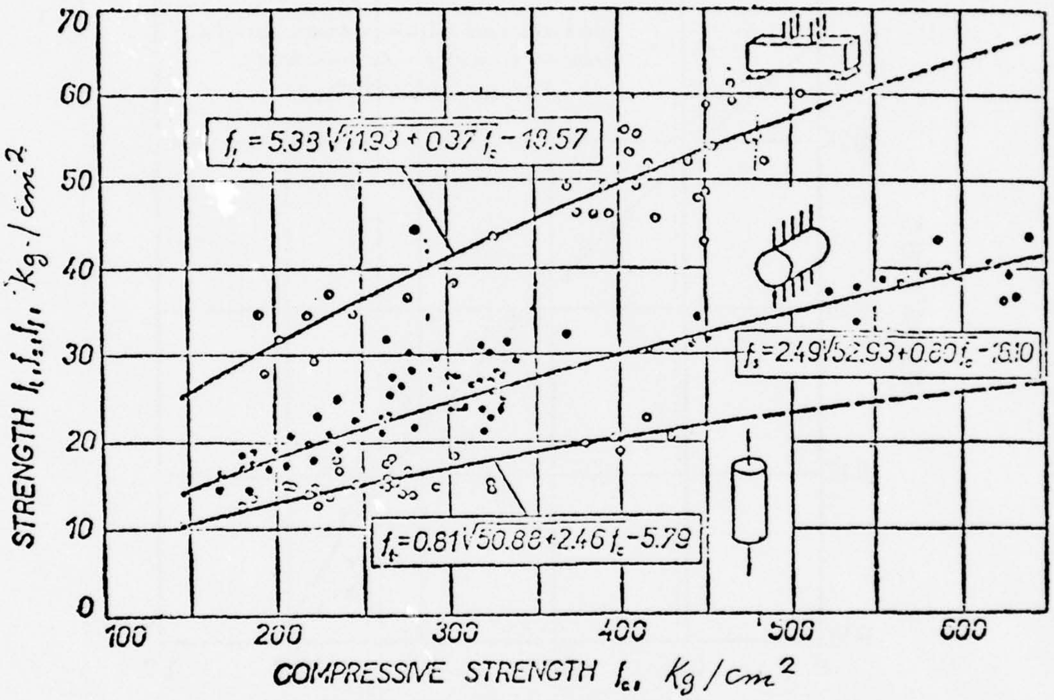

Figure 3.6. Concrete with river pebbles--Mix I; cylinders 15 cm in diameter by 30 cm long (6 by 12 in.), age 28 days; direct tensile strength f_t , splitting strength f_x , and flexural strength f_f of prisms 15 by 15 by 70 cm (6 by 6 by 28 in.) as functions of compressive strength f_e , in tubes 20 cm (8 in.) on a side. Each point represents the result of one test (after Kadlecek and Spetla[3])

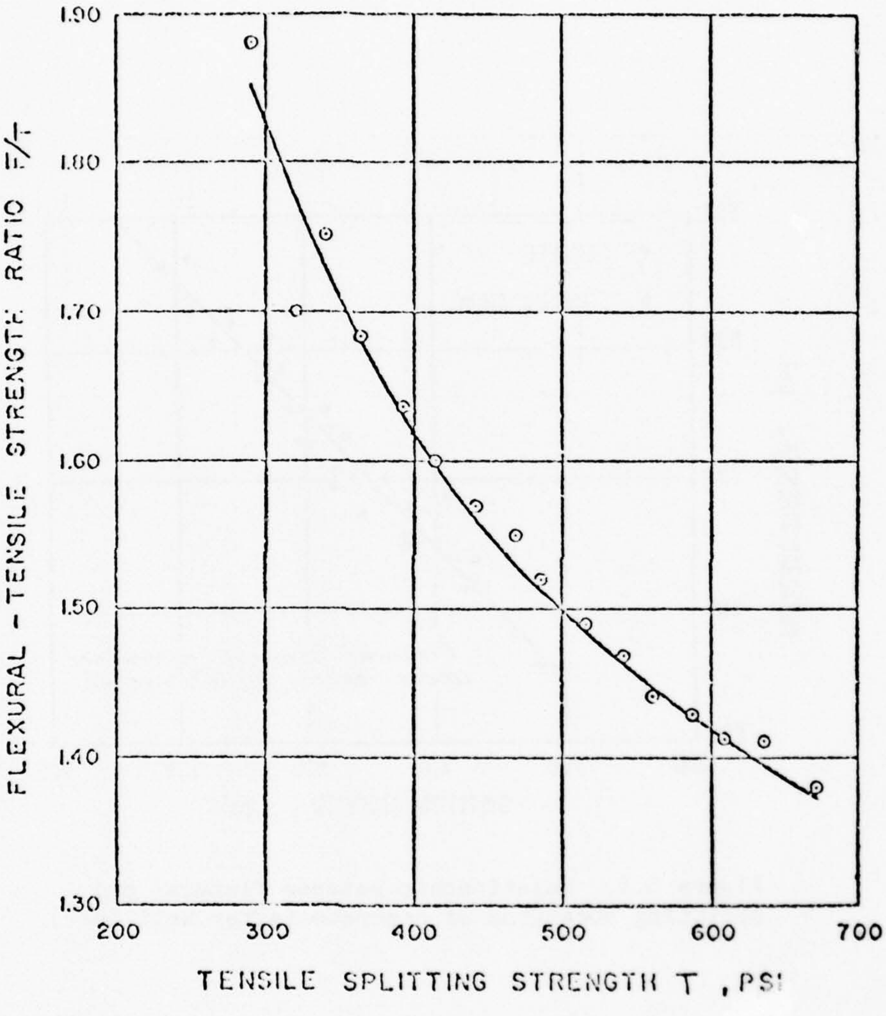

Figure 3.7. Ratio of flexural to tensile splitting strength versus tensile splitting strength of concrete (after Narrow and Ulberg[35])

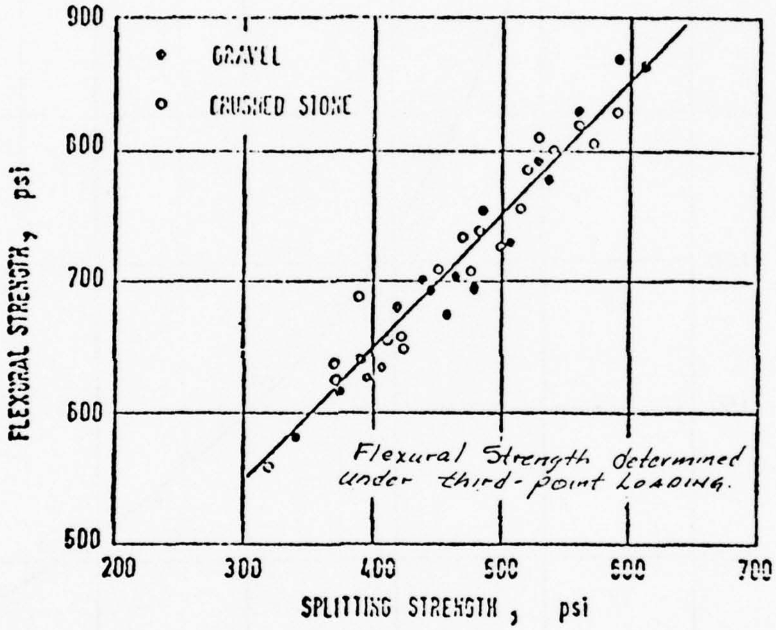

Figure 3.8. Relationship between flexural and splitting strengths of concrete (after Neville[15])

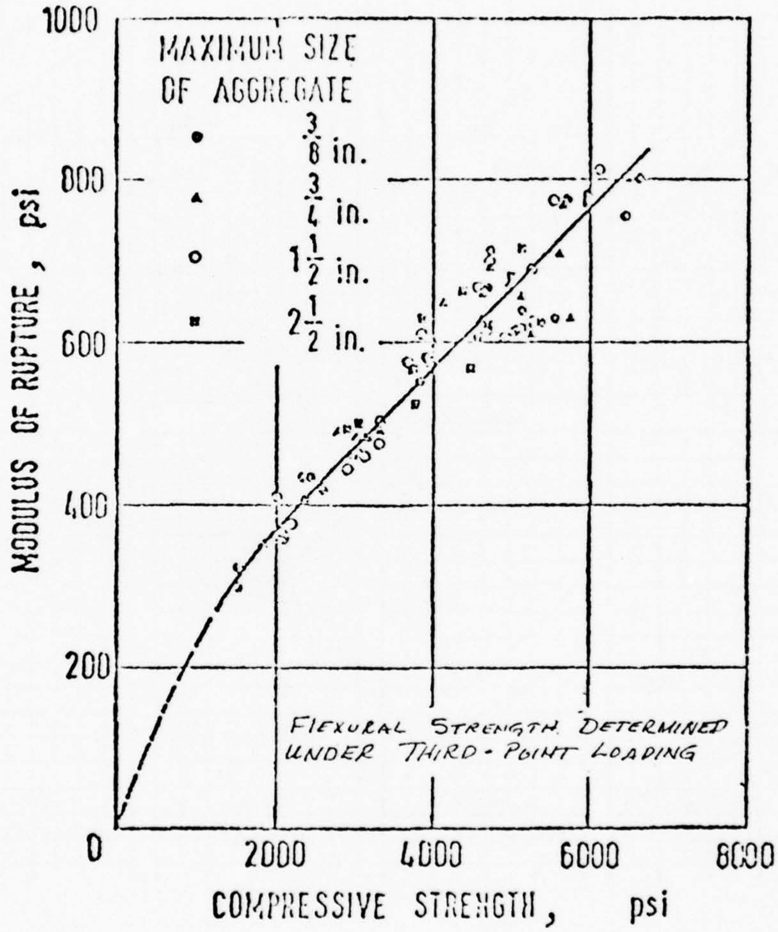

Figure 3.9. Relationship between compressive and flexural strengths of concrete (after Neville[15])

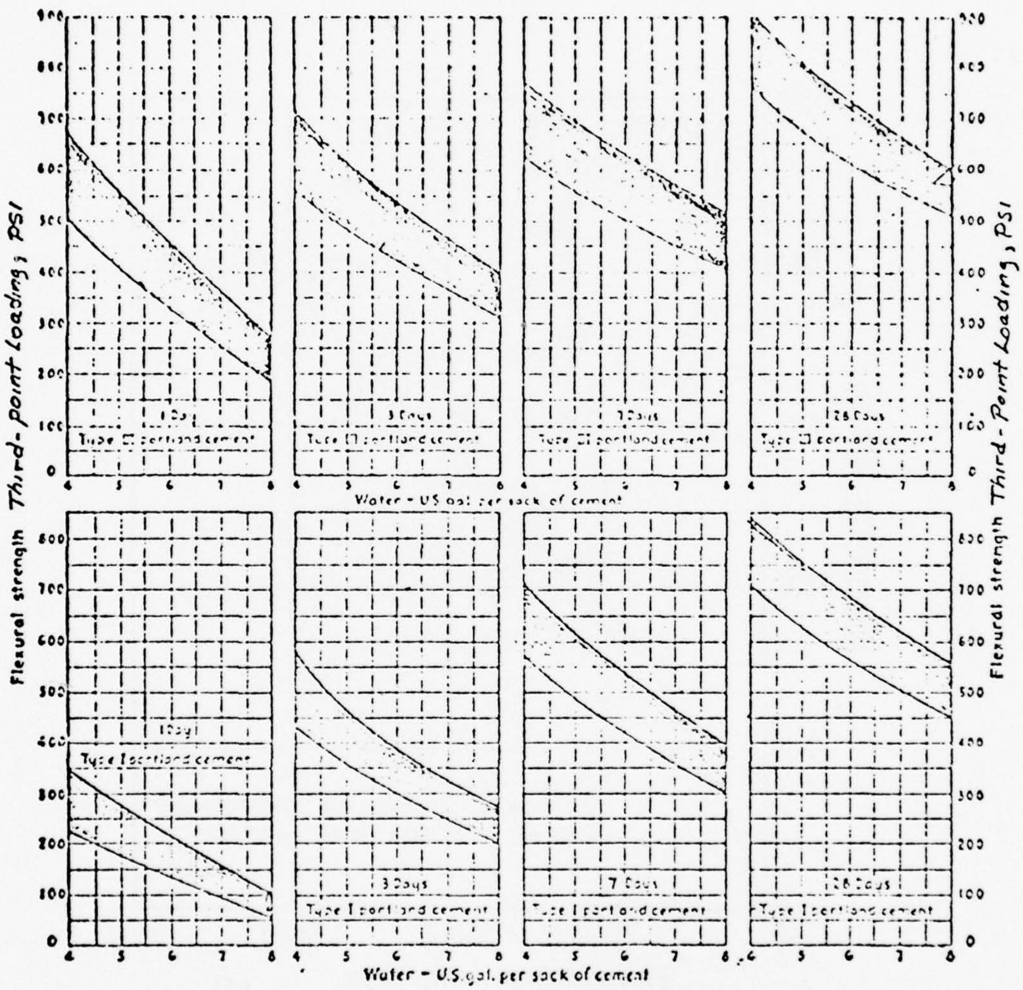

Figure 3.10. Age-flexural strength relationship for types I and III portland cements (after PCA37)

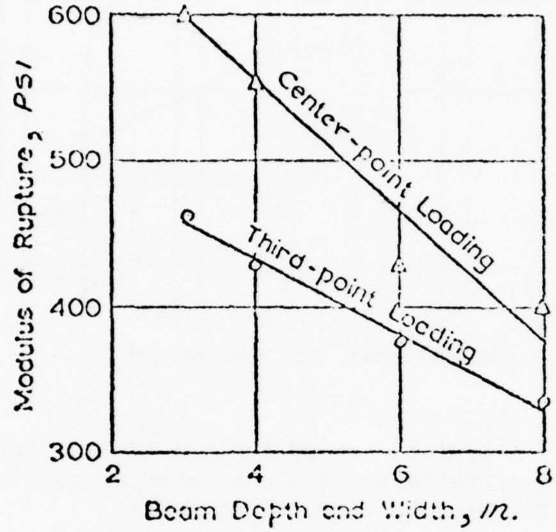

Figure 3.11. Modulus of rupture of beams of different sizes subjected to center-point and third-point loading (after Neville[4])

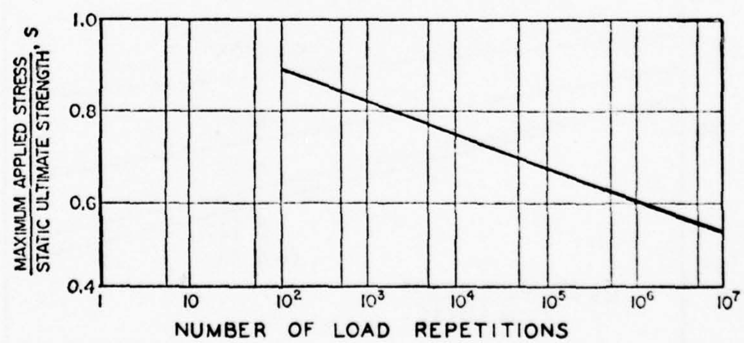

Figure 3.12. Typical S-N (stress versus log of number of repetitions) curve

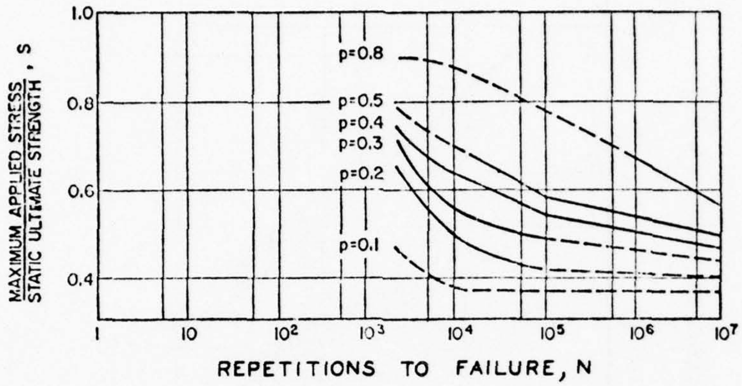

Figure 3.13. A typical set of fatigue curves for probabilities of failure for plain concrete subjected to reversed flexural loading (after McCall[55])

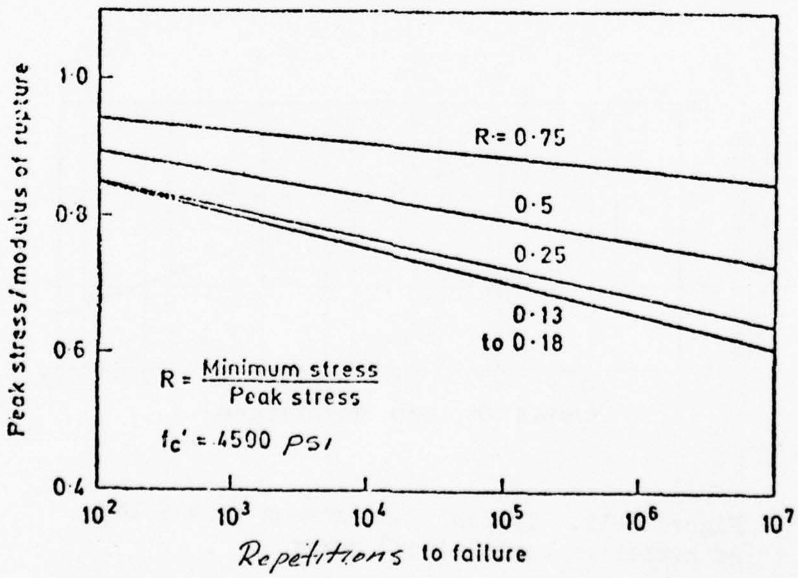

Figure 3.14. Effect of range of stress on fatigue life of concrete beams (after Murdock and Kesler[59])

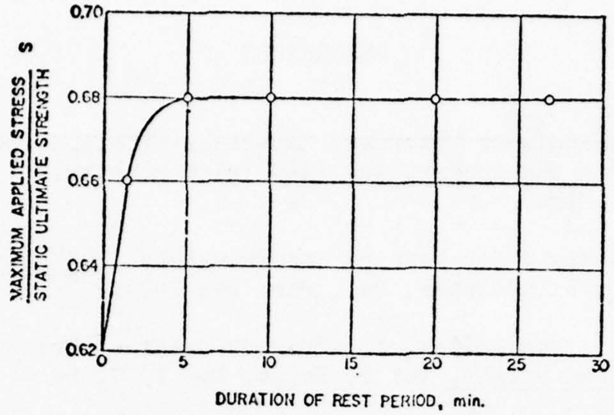

Figure 3.15. Effect of rest periods on fatigue strength at 10 million repetitions (after Hilsdorf and Kesler[61])

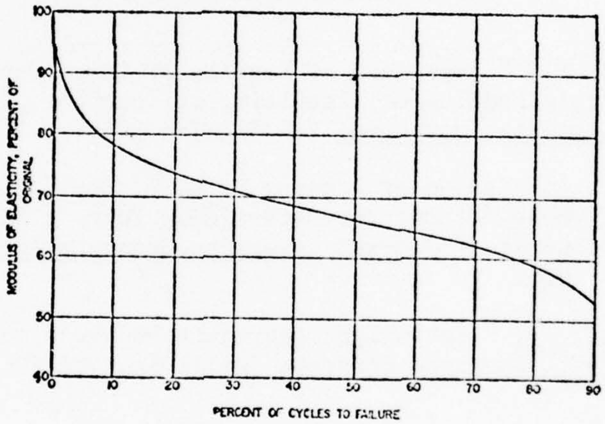

Figure 3.16. Reduction in secant modulus of elasticity as the result of repeated loads (after Linger and Gillespie[57])

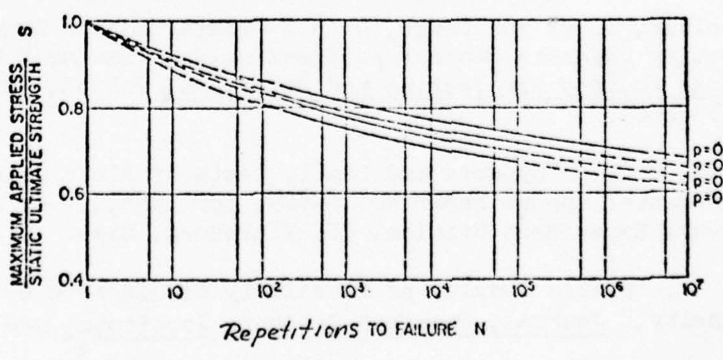

Figure 3.17. Probabilistic plain concrete failure curves for use with Miner's hypothesis (after Hilsdorf and Kesler[61])

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NOTATION

a	Constant; also, distance between the line of fracture and the nearest support measured along the center line of the bottom surface of the beam, in.
A	Cross-sectional area
b	Average width of specimen, in.
c	Distance from neutral axis to farthest fiber (one-half the depth of the beam), in.
C	Factor which depends on shape and size of specimen, mode of vibration, and Poisson's ratio
d	Density; also, average depth of specimen, in.; also, diameter of cylinder, in.
D	Deflection
e	Total deformation
E	Young's modulus of elasticity
E_c	Static modulus of elasticity of the concrete, psi
f_e	Compressive strength
f_f	Flexural strength
f_t	Direct tensile strength
f_x	Splitting strength
f'_c	Compressive strength of the concrete at time of test, psi
F	Flexural strength
G	Modulus of rigidity
h	Height of beam
I	Moment of inertia, $in.^4$
k	Pickett's correction for shear (third-point loading)
ℓ	Span length, in.; also, length of cylinder, in.
L	Length of specimen; also, span length
M	Maximum bending moment, in.-lb
n	Resonant frequency
N	Number of load repetitions; also, number of load repetitions to failure
P	Applied load, lb; also, probability of failure
R	Flexural strength; also, modulus of rupture, psi
s_1, s_2	Alternating stresses

S	Stress = maximum applied stress/static ultimate strength
T	Tensile strength, psi; also, tensile splitting strength, psi
V	Compressional wave velocity
w	Width of beam; also, air-dry weight of the concrete at time of test, pcf
W	Weight of specimen
ε	Unit of deformation
ε_1	Longitudinal strain produced by the stress at 40 percent of the ultimate load
ε_{t1}	Transverse strain at midheight of specimen produced by a stress corresponding to a longitudinal strain of 50 μin./in.
ε_{t2}	Transverse strain at midheight of specimen produced by a stress corresponding to 40 percent of the ultimate load
ν	Poisson's ratio
σ	Unit stress

CHAPTER 4: GRANULAR MATERIALS

4.1 INTRODUCTION

Granular aggregates are essential pavement materials used commonly as base and subbase courses in flexible pavements and as subbase in rigid pavements. The strength of a flexible pavement is derived from the distribution of load over the subgrade through the subbase, base, and surface courses, while in a rigid pavement suitable distribution can be provided by the surface layer alone or with a well-constructed subbase layer. A base course is a layer of granular material which lies immediately below the bituminous concrete surfacing layer of a flexible pavement, whereas a subbase is a layer of material between the base* and subgrade. The granular material under a rigid pavement is commonly termed the subbase.

Since the base course lies close to the pavement surface, it must possess high resistance to deformation to withstand the high tire pressures imposed on the pavement, and since subgrades normally do not have sufficient strength to withstand heavy concentrated loads, base and subbase courses are required to distribute applied loads over a larger area of the subgrade. The base course must be stable under the load and commonly is constructed of gravel, crushed stone, slag, or similar high-quality processed aggregates. On the other hand, a subbase can be of a lower quality and generally consists of locally available materials.

The function of the base and subbase courses varies according to the type of pavement. Under a flexible pavement, base and subbase courses are used primarily to increase the load-supporting capacity of the pavement by distributing the load through a finite thickness of pavement, and may also provide drainage and give added protection against frost action when necessary. The strength of a rigid pavement is derived from the load-carrying capacity of the pavement as a whole, and thus the

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* This may be a bituminous base course.

purposes of a subbase layer of selected granular material under the concrete slab are more to prevent mud pumping, reduce frost damage, improve drainage, control moisture in subgrade soils having high volume changes, and improve the constancy and effectiveness of pavement support than to increase the structural capacity.

4.2 PROPERTIES OF GRANULAR MATERIALS

The behavior of granular materials is influenced by various properties of the constituent particles which are discussed below.

4.2.1 DENSITY AND GRADATION

The following material is quoted from the Highway Engineering Handbook:[1]

> The stability of a granular base course depends upon particle-size distribution, particle shape, relative density, internal friction, and cohesion. A granular material designed for stability should exhibit high internal friction to resist deformation under load. Internal friction and subsequent shearing resistance depend to a large extent upon density, particle shape, and grain-size distribution. Of these latter two factors, the size distribution of the aggregate is the most important. This is particularly true when considering the quantity of fines which is in the aggregate mixture.
>
> An aggregate which contains little or no fines and is well graded develops stability primarily from grain-to-grain contact. This type of material has a relatively low density, is nonfrost-susceptible, and is pervious. However, this material is very difficult to handle during construction, because of its noncohesive nature.
>
> An aggregate which contains sufficient fines to fill the voids between coarse-aggregate particles develops strength from grain-to-grain contact, with increased resistance against deformation due to the increased density. The density of such a mixture is high, and it is practically impervious, but it may be frost-susceptible. This material is moderately difficult to compact but is ideal from the standpoint of stability for it has relatively high shearing resistance either in a confined or unconfined condition.
>
> At the other extreme, a material which contains excessive amounts of fine materials has no substantial grain-to-grain contact. The density is low, it is practically impervious,

and it is frost-susceptible. Such material may be easy to manipulate during construction.

Extensive tests were performed in the laboratory of the National Crushed Stone Association on graded aggregate mixtures.[2-6] Kalcheff[2] emphasized the effect of maximum size on rigidity of the aggregate. This is illustrated in Figure 4.1. The tested aggregates came from the same source but had different nominal sizes. It was noted that the rigidity of the aggregate material can be improved by increasing the nominal aggregate size. Additional insurance against permanent deformation could therefore be achieved by using a larger nominal size. Figure 4.2 illustrates the importance of proper gradation and demonstrates that there exist certain optimum gradations as related to ultimate density and strength. In the series of tests illustrated, the maximum size was held constant but total gradation was varied. The percentage of minus No. 200 material was used to identify each continuous gradation. Note that the gradation necessary for maximum density was not necessarily the same as the gradation needed to achieve maximum strength.

4.2.2 PLASTICITY

The physical properties of the soil binder have a great effect on stability when grain-to-grain contact of the coarse aggregate is destroyed. Test results have shown that the strength of soil decreases as the soil-binder plasticity increases; this effect is most pronounced for high soil content percentages. Thus, an added specification limiting the plasticity of a soil should be used. The AASHTO specifications set the liquid limit at 25 percent and plasticity index at 6 percent. If the quantity of binder, however, is controlled within close limits to a value equal to or less than the optimum amount, plasticity becomes a secondary consideration. For aggregate surface courses, it is desirable to use more binder to provide some cohesion to the mass. Here the plasticity is important.

Figure 4.3 is a typical example of the large reduction in the ultimate axial stress caused by changes in the plasticity of the fine fraction (minus No. 40 mesh) reported by Kalcheff.[2] The overall gradations of these mixtures were identical.

The addition of plastic fines to graded aggregates not only decreases the load-bearing capacity, but also greatly reduces their rigidity. This is demonstrated in Figure 4.4 by Kalcheff[2] which illustrates a typical example of the loss in rigidity of a graded aggregate base to be expected by an increase in the plasticity of its fine fraction. Reducing the plasticity to zero can, therefore, reduce substantially the potential for permanent deformation or rutting.

4.2.3 PERMEABILITY

Thompson[7] studied the effect of pore pressures in granular layers on the behavior of flexible pavements. He found that the amount of fine material is particularly important, and that there seems to be an optimum amount of fines for maximum stability, and a different optimum amount for maximum density. Furthermore, the amount of fines affects the behavior of saturated granular layers. Test results showed that granular materials with a degree of saturation above about 80 percent deteriorate rapidly under repeated loading. The effect of the degree of saturation was significant because of the development of excess pore water pressures at high moisture contents. The pore pressure measurements in the laboratory studies were as high as 0.15 psi at stress levels which were typical of the values found in pavements. Thompson stated that even apparently small pore pressures could reduce the effective stress in the base course to values approaching failure. When liquefaction occurs, the fine material is free to go into suspension and move away from the loaded area. This may explain the change of thickness of the granular layers in a wheel path in the AASHTO road test. He further concluded that besides the loss of subgrade support during the wet season, the saturation of the granular materials and consequent liquefaction of these materials also can attribute to the spring breakup of the pavement.

4.2.4 STATIC PROPERTIES

Holz and Gibbs[8] performed static triaxial tests on free-draining gravelly soils to determine factors affecting their shear strength. It was reported that:

a. The rate of shear had no effect.

b. An increase in the relative density produced a higher angle of internal friction and therefore a higher shear strength.

c. An increase in gravel content (up to 50 percent) produced an increase in the angle of internal friction and therefore a higher shear strength.

d. The maximum particle size range of 3/4 to 3 in. produced no appreciable change in the shear strength.

4.2.5 RESILIENT PROPERTIES

Although repeated load tests have been performed on cohesive soils for nearly two decades, it has only been in the last 10 years that this type of test has been used to any large extent to study the resilient characteristics of granular materials. The consensus from these studies has been that the response of granular materials to repeated loading is different from their response to static loading. The factors influencing the resilient responses of granular materials are discussed below.

4.2.5.1 Laboratory Studies. Many investigators have studied the resilient behavior of granular materials subjected to repeated stresses in the triaxial apparatus. The general consensus of the studies appears to be that the following factors may have a significant influence on the stress-deformation characteristics under short-duration repeated loads:

a. Stress level (confining pressure).

b. Degree of saturation.

c. Dry density (or void ratio).

d. Fines content.

e. Stress duration and frequency.

These factors are discussed separately in the following passages. This discussion is mostly taken from Hicks' dissertation.[9]

4.2.5.1.1 Stress Level. Studies of resilient response of sands and gravels subjected to repeated axial stresses have all indicated that the resilient modulus increases with confining pressure and is relatively unaffected by the magnitude of the repeated deviator stress, so long as the repeated stress does not cause excessive plastic deformation.

Biarez[10] has presented results of triaxial compressive cyclic load tests on a uniform sand (grain diameter of 0.016 in.) in which the variation of the modulus of resilient deformation with mean normal stress was investigated. From the results obtained after several cycles of load, he concluded that the variation of the modulus with the mean normal stress may be stated as

$$E = K_1 \sigma_m^{K_2} \tag{4.1}$$

in which

E = modulus of elasticity

K_1 = constant

σ_m = mean normal stress; i.e., (sum of principal stresses)/3

K_2 = exponent varying from 0.5 to 0.6

Trollope, Lee, and Morris[11] subjected a sand to slow repeated cyclic loads and found that the modulus increased with increasing confining pressure, but was unaffected by the axial stress so long as a failure condition was not reached.

The Texas Transportation Institute has also investigated the behavior of granular materials in repeated loading. Based on the results of tests on partially saturated well-graded aggregates, Dunlap[12] has suggested an equation of the form

$$M_z = K_2 + K_3(\sigma_r + \sigma_\theta) \tag{4.2}$$

in which

M_z = modulus of deformation measured in the direction of an applied stress σ_z

K_2 = modulus of resilient deformation for the unconfined condition

K_3 = constant of proportionality

σ_r and σ_θ = radial and tangential stresses, respectively

Dependence of the resilient modulus on stress level has also been observed at the University of California (Mitry,[13] Shifley,[14] and

Kasianchuk[15]) and by The Asphalt Institute (Kallas and Riley[16]). Results obtained from repetitive load triaxial tests were expressed in terms of either Equation 4.2 or by

$$M_r = K_1 \sigma_3^{K_2} \tag{4.3}$$

where σ_3 is the confining pressure and K_1 and K_2 constants.

In repeated load triaxial tests on sands, Morgan[17] was one of the first investigators to obtain direct measurements of both axial and radial deflections. These were made to calculate the resilient modulus and Poisson's ratio. It was found that M_r depended mainly on the level of confining pressure, although there was a slight decrease with increasing deviator stress. Poisson's ratio did not appear to be related to confining pressure or deviator stress and varied from 0.20 to 0.40 over the range in stress levels employed (confining pressures of 10 to 30 psi and deviator stresses of 20 to 50 psi).

Experimental data are also available which show the effect of various stress states on the dynamic shear modulus of dry sand measured by vibration and on the static shear modulus measured by repeated torsion tests (Hardin and Black[18]). In these tests, the sand stiffness was found to vary with the sum of the static normal stresses θ. In all cases, the stiffness was independent of the deviatoric component of the initial static state of stress and the rate of loading.

Based on laboratory repeated load tests, Hicks[9] concluded that the resilient properties of untreated granular materials are affected most significantly by stress level. In all cases, the modulus increased considerably with the confining pressure and slightly with the repeated axial stress. So long as shear failure does not occur, the modulus can be approximately related to the confining pressure σ_3 or to the sum of principal stresses according to

$$M_R = K_1 \sigma_3^{K_2} \tag{4.3 bis}$$

and

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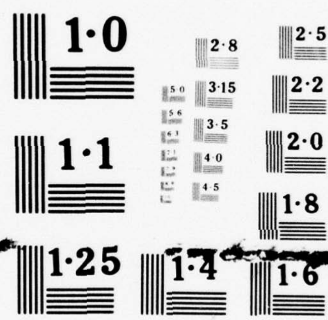

NATIONAL BUREAU OF STANDARDS
MICROCOPY RESOLUTION TEST CHART

$$M_R = K_1' \theta^{K_2'} \quad (4.4)$$

Poisson's ratio increased with decreasing confining pressure and increasing repeated axial stress where the change in Poisson's ratio could be approximated as follows: $\nu = A_0 + A_1(\sigma_1/\sigma_3) + A_2(\sigma_1/\sigma_3)^2 + A_3(\sigma_1/\sigma_3)^3$. Figures 4.5 and 4.6 show the effect of the axial stress on the resilient modulus. Figure 4.7 shows the variation in Poisson's ratio with stress level.

Allen[19] conducted a series of laboratory repeated load tests on a variety of granular materials. Unlike all other laboratory tests reported in this report, the triaxial chamber confining pressure was varied simultaneously with the axial load to simulate the actual stress pulse in flexible pavements. He found that the testing variable most significantly affecting the resilient response of the granular specimens was the applied state of stress. The effects of material type on the resilient parameters are slight compared with the effects of changes in the state of stress. In general, the crushed stone yielded slightly higher values of resilient modulus than the gravel. The modulus of a blend of the gravel and limestone was normally between those of the other materials. Poisson's ratio varied only minimally from one material to another, and the values calculated for the gravel normally exceeded those for the crushed stone.

4.2.5.1.2 Degree of Saturation. Studies concerned with the resilient response of sands and gravels at different degrees of saturation (or water contents) have generally indicated that the resilient modulus decreases as the degree of saturation increases, so long as comparisons are made on the basis of total confining pressures. Comparisons on the basis of effective stresses indicate that the resilient moduli for 100 percent saturated samples differ only slightly from those for dry samples.

Studies involving the repeated load effects in granular materials have been conducted at Purdue University. Johnson[20] and Johnson and Yoder[21] performed laboratory triaxial tests on soil-sand mixtures and measured pore water pressures and volume changes. An initially high

pore water pressure gradient reached equilibrium after about 1000 cycles of load, and volume changes produced high total permanent deformations and eventual failure. The stresses were varied from 30 to 40 psi.

Haynes and Yoder[22] presented the results of undrained repeated-load triaxial-compression tests on gravel and crushed stone, similar to the type of materials used for the base course in the AASHTO Road Test. The effect of saturation was shown to be significant. It indicated that there is a critical degree of saturation (about 80 percent) above which granular materials become unstable and deteriorate rapidly when subjected to repeated loading.

Coffman, Kraft, and Tomayo[23] have determined complex moduli for the granular materials representing both the subbase and base course at the AASHTO Road Test, as well as the subgrade soil noted earlier. Over a limited range of water contents and densities, the complex modulus increased slightly with increased dry density and decreased slightly with increased water content for both base and subbase materials.

Studies at the University of California have yielded similar results. Mitry[13] and Seed et al.[24] reported tests on a well-graded gravel in both the dry and saturated conditions under repeated triaxial loading. In terms of effective stress, it was found that the resilient moduli of saturated specimens were slightly higher for drained tests and nearly the same for undrained tests than values from corresponding tests on dry specimens. Shifley[14] tested a well-graded crushed aggregate and found that the coefficient K_1 in Equation 4.3 was reduced from 13,500 for the dry case to 9,300 for the partially saturated case, while K_2 was unchanged. (It should be mentioned that the partially saturated modulus is expressed in terms of total stresses.) Kasianchuk[15] also subjected a saturated aggregate subbase material to repeated stresses. The testing was carried out undrained and static and transient pore pressures measured throughout the test. It was reported that with increasing number of repeated loads, an excess pore water pressure tended to develop, which resulted in a reduction of the effective confining stress $\overline{\sigma}_3$ and the resilient modulus. Though the conditions in

the test were extreme in the sense that they will probably not occur in a pavement, it does indicate the potential of a reduction in the modulus when the pavement is saturated.

Morgan[17] reported results of repeated load tests on two sands and found that the behavior of the free-draining saturated sands were only slightly different from tests on air-dried sands. There was a tendency, however, for the saturated samples to show larger permanent and recoverable strains even though they were tested at higher densities.

Repeated load tests made by The Asphalt Institute also showed a reduction in the modulus with increased water content (degree of saturation). Kallas and Riley[16] found that as the water content increased from 2.4 to 8.2 percent, the coefficient K_1 in Equation 4.3 was reduced from 10,618 to 8,687, while the coefficient K_2 remained essentially constant. For a confining pressure of 10 psi, this would mean a decrease in modulus from about 32,000 to 22,000 psi, or about 30 percent. Tests on aggregate base materials from the San Diego Test Road showed similar trends (Hicks and Finn[25]). Expressing the results in terms of Equation 4.4, the constant K_1' dropped from 5,400 to 2,100 as the water content was increased from about 2.7 to 6.3 percent. Again K_2' remained constant.

Thompson[7] reported results of repeated load triaxial tests on the crushed stone from the AASHTO Road Test at varying degrees of saturation. Measurements of permanent and resilient deformation as well as transient pore pressure were taken throughout the test. The samples were tested in their initial state (initial degree of saturation from 10 to 70 percent) and after they had been soaked in water (final degree of saturation from 79 to 100 percent). In all cases, the samples experienced a substantial increase in permanent deformation after soaking. It was suggested that one reason for the observed increase was development of transient pore pressures in the soaked samples. No pore pressure was measured in the unsoaked specimens, but pressures of the order of 0.05 to 0.15 psi were measured in the soaked tests for repeated stresses of 40 and 60 psi. Although the effect of the transient pore pressure on the resilient properties of the granular base was not reported, it should be small if pore

pressures of the order of 0.05 to 0.15 psi are typical of those found in most granular materials.

Repeated load tests conducted by Hicks[9] showed that in all cases K_1 decreased from the dry to partially saturated test series where the comparisons were made on the basis of total stresses. Figures 4.8 provides an indication of this effort for each aggregate at two levels of grading. The relationship between Poisson's ratio and the principal stress ratio is shown in Figure 4.9. In general, Poisson's ratio decreases with increasing degree of saturation.

4.2.5.1.3 Dry Density. Although studies showing the effect of density (or void ratio) on the resilient properties of granular material are limited, the general conclusion is that dry density has a significant effect. In general, as the density increases (or void ratio decreases), the resilient responses are reduced for the same magnitude of stress.

Trollope, Lee, and Morris[11] reported the results of slow repeated cyclic tests on a poorly graded sand. The study indicated that the modulus of the sand increased with a decrease in void ratio (increase in dry density). The difference in moduli between a loose and dense sand was reported to be as much as 50 percent. Similar findings were published by Coffman, Kraft, and Tomayo.[23] For the range of water content and density investigated, the complex modulus of both an aggregate base and subbase increased with density. For an increase in dry density from 136 to 140 pcf, the modulus increased by as much as 27 percent.

Some repeated load triaxial tests have been conducted by Johnson[26] on air-dried, fairly rounded, well-graded gravel. In these tests, the influences of void ratio and confining pressure were investigated. The most significant result was the striking dependence of the modulus of resilient deformation on the confining pressure.

In the laboratory repeated triaxial compression tests conducted by Hicks,[9] he found the coefficients K_1 and K_1 (Equations 4.3 and 4.4) to increase with increasing density, while K_2 and K_2 remained relatively constant or decreased slightly. The effects are shown for the partially crushed aggregate (dry test series) in Figure 4.10. Similar trends were found to exist for the partially saturated and saturated

test series. For the crushed aggregate, K_1 also tended to increase with density. It is worthwhile to point out again that K_2 remained nearly constant.

In the repeated load triaxial tests with lateral stresses, Allen[19] found that the resilient parameters are affected by variations in the dry density of the specimen. Generally, the resilient modulus increases as density increases. Poisson's ratio showed no consistent variation with changes in density. The values of resilient Poisson's ratio were very similar for all specimens at corresponding values of σ_1/σ_3 for the variable confining pressure test.

4.2.5.1.4 Fines Content. Studies demonstrating the variation in response of granular materials subjected to repeated axial stresses indicate that the fines content (percent passing No. 200 sieve) can also affect the resilient behavior. The actual effect is unclear, although it does appear that the magnitude of the change is dependent on aggregate type.

Haynes and Yoder[22] presented results of repeated load triaxial tests on a gravel and crushed stone for a range of minus No. 200 sieve material. Typical results indicated that for the stress conditions used (σ_d = 55 psi, σ_3 = 15 psi) the modulus is only slightly affected by grading. For the gravel, the resilient modulus was generally lower for the middle level of fines (9.1 percent), while for the crushed stone it was essentially the same regardless of the grading.

On the other hand, Barter[27] presented data on the effects of the permeability of granular bases which demonstrated that materials with comparatively high fines contents (more than 5 percent passing the No. 200 sieve) may require some time to drain, causing a potential development of pore pressure under load. According to Barter, this tended to reduce the strength of the base layer to some critical level. Similar observations were reported by Thompson[7] on repeated load tests of the crushed stone from the AASHTO Road Test. Tests on soaked specimens indicated that the samples with high fines contents experienced greater permanent deformation than blends with low fines. For example, increasing the fines from 3 to 21 percent caused an increase in the total permanent deformation of more

than 100 percent. Limited data for the same samples in the unsoaked condition showed the same trend.

Results of laboratory repeated load tests conducted by Hicks[9] revealed that the resilient Poisson's ratio was also influenced by the fines content. In most instances, as the fines content increased, the mean value for Poisson's ratio was reduced. An example of the manner in which Poisson's ratio varied with grading for partially crushed and crushed aggregate is shown in Figure 4.11. Regardless of aggregate type, there was a noticeable reduction as the percent fines increased and the reduction appeared to be a function of the aggregate type; i.e., there was a greater reduction for the crushed aggregate.

4.2.5.1.5 Stress Duration and Frequency. In spite of the limited research, there appears to be some effect of stress duration and frequency on the response of granular materials. The magnitude of the effect on resilient moduli is relatively uncertain. In some cases, only slight changes were observed, while in other investigations, changes as great as 100 percent were noted.

Seed and Chan[28] investigated the effect of the duration of stress on the total deformation of soil specimens subjected to repeated loading. An increase in the duration of stress application, for intervals up to 2 minutes, resulted in an increase in the total deformation of a silty sand. From their data, it is also possible to show that the modulus of resilient deformation increases as the duration of load application decreases and that this increase is more pronounced for very short durations of load. However, considering the range of values investigated, the change was relatively small.

In the laboratory repeated triaxial tests with both variable and constant confining pressures, Allen[19] reported that the resilient response of well-graded granular materials is independent of stress pulse duration. Therefore, any pulse duration in the range of those applied to elements of pavements by wheel loads moving at speeds of about 15 to 70 mph may be used in laboratory investigations.

Contrary results of work were reported by Coffman, Kraft, and Tomayo.[23] They reported that in all cases higher modulus values were

observed at the higher frequency. These increases ranged from 50 to 100 percent, depending on water content and dry density. It should be emphasized that the modulus values reported were determined from static tests and transformed into the frequency domain.

In repeated tests on a limestone gravel, Armstrong (as reported by Dunlap[29]) related the influence of frequency and duration of repeated loads to total deformation. He found that long loading durations and short intervals produce larger deformations for any number of stress repetitions.

Many other investigators have studied the general properties of granular materials, i.e., gradation, moisture content, density angularity, and porosity, on the resiliency of granular materials. Hveem et al.[30] investigated the resilience characteristics of granular base and subbase materials, as well as fine-grained subgrade soils, using the resiliometer. He found the resilience value at a given pressure decreases as the quality of the granular material increases. Barkan[31] concluded, on the basis of laboratory tests conducted on sand confined in a consolidation mold and subjected to repeated loading, that Young's modulus for a pure sand is a stable characteristic that does not change much with changes in moisture content, grain size, or porosity of the sand. White[32] determined the effect of gradation, moisture content, compaction, and applied load on the resiliency of base course materials. Load applied to materials compacted in a metal cylinder 6 in. in diameter was held constant until deformation ceased, at which time deflection was measured. The load was then released and the rebound readings recorded. He concluded that the resiliency after 50 cycles of loading and unloading does not appear to be affected by the type of aggregate, gradation, moisture content, or density of the original specimen, but that the resilient deflection would appear to be dependent on both the magnitude and the number of applications of the load.

Brown[33] conducted a series of laboratory repeated triaxial tests on a crushed granite. He found that under drained conditions, the resilient strain reached equilibrium values after approximately 10^4

cycles of deviator stress. The resilient strain at equilibrium was related to the applied stresses by the equation

$$E_r = 237 \left[10^{-6} \left(\frac{\theta}{\sigma_d' + 0.25\sigma_3} \right)^{1.54} \right] \tag{4.5}$$

where θ and σ_d' are mean normal stress and effective deviator stress. Seed et al.[34] summarized many investigations discussed in the preceding paragraphs, and they are tabulated in Table 4.1.

The laboratory repeated load tests conducted by Hicks[9] may best summarize the factors influencing the resilient properties of granular materials. He concluded that the resilient properties of granular material were also affected by factors such as aggregate density, aggregate gradation (percent passing No. 200 sieve), aggregate type, and degree of saturation. At a given stress level, the modulus increased with increasing density, increasing particle angularity or surface roughness, decreasing fines content, and decreasing degree of saturation. Poisson's ratio, however, was slightly influenced by density, generally decreasing as the fines content increased, and generally decreasing as the degree of saturation increased. Small changes in the modulus relationship or Poisson's ratio for the granular base layer can result in significant changes in the response of the pavement structure.

4.2.5.2 Field Studies. A few studies were conducted to investigate the performance of granular base materials under conditions of loading representative of the actual pavement structure. The information which exists points to the importance of better defining the characteristics of these materials. In those studies which have investigated the role of the granular base in the behavior of the total pavement system, it appears that for a given base type, grading, and density, the response of pavements is most affected by the degree of saturation in the base and the type of loading conditions. Each of these factors is discussed below; the discussions are taken from Hicks.[9]

4.2.5.2.1 Degree of Saturation. Results of field tests have been reported for both prototype and full-scale pavements. On the basis

of these studies, it appears that both the resilient and total deformations are influenced by the degree of saturation. However, the actual effect on each appears to be governed by the type of material and level of density.

At the AASHTO Road Test, 80 percent of the failures of the flexible pavements occurred during the spring. This is particularly significant when it is noted that failures of rigid pavements in the AASHTO Road Test[35] were distributed uniformly throughout the year. Similar results were reported at the WASHO Road Test.[36] This deterioration is normally attributed to reduced subgrade strength; however, at the AASHTO Road Test, there was only a small change in the moisture content and the resilient deformation (Table 4.2) of the subgrade during the spring of the year. Another factor which was observed was that only a small part of the total rut depth occurred in the subgrade, while more than 50 percent was indicated to have occurred in the base and subbase layers (Table 4.3). Seasonal changes in moisture content of the base and subbase layers ranged on the order of 1 to 2 percent maximum. Although this does not appear significant, the small change in water content represents an increase in the degree of saturation from about 60 to 80 percent. Because the degree of saturation increased to 80 percent or higher during the spring, it was suggested that this led to the relative instability and rutting in the granular layers reported by Haynes and Yoder.[22]

At this point, it is interesting to note that Barber and Steffens[37] suggested that critical pore pressures which affect the strength of a base course can be controlled by changing the gradation and drainage so that the material when compacted has void spaces which are less than 80 percent saturated. Their analysis of field measurements at the Hybla Valley Test Road indicated that positive pore pressures could develop under these conditions, and these could lower the static bearing capacity of the pavement structure to some critical level. They demonstrated that a temperature change of 6.5° F in a 12-in. base layer could produce a rise in the effective water table of 18 in., or an increase in the pore pressure of about 0.54 psi.

Thompson[7] reported results of repeated plate tests on a small model (dimensions 6 in. wide by 51 in. long by 18 in. deep) where the influence of factors such as subgrade moisture and density and base moisture and density were studied. The water level in the model could be controlled as could the combinations of the pavement layers. When all the results were evaluated, it was shown that the degree of saturation of the base was not related to total deformation. However, at specific levels of base density, there was a definite increase in permanent total deformation with an increase in the degree of saturation. These results compared well with those obtained on the Pavement Test Track developed at the University of Illinois. This facility, also designed so the water table could be raised and lowered, enabled pavements of various thicknesses to be constructed and then subjected to repeated wheel loads. Typical results indicated that the pavements performed satisfactorily until they were soaked and then they deteriorated rapidly. The deterioration was not due to saturation of the subgrade, since there was no significant moisture change during soaking. It was concluded that the deterioration was due to the increase in degree of saturation of the base layer, which occurred with increased repetitions. Good agreement was found when the 80 percent criterion suggested by Barber and Steffens[37] for critical degree of saturation was used to evaluate the different test sets.

Hicks[9] made a series of repeated plate load tests on a prototype pavement and found that for a given stress level the resilient surface deflections increased with increasing degree of saturation of the base layer. However, the deflection at the subgrade-base interface did not change with degree of saturation. For this pavement, the increase in surface deflection with degree of saturation (15 to 20 percent) was due entirely to a reduction in the stiffness of the granular base.

4.2.5.2.2 Loading Conditions. Results of repeated plate load tests reported by Mitry[13] and Shifley[14] indicated the presence of stiffening type relations between plate pressure and surface deflection for pavements composed of bituminous concrete, granular base, and clay subgrade. This behavior was attributed to the increased modulus within the

base as the plate pressure increased and was consistent with the laboratory behavior described previously. For tests with the larger plates, the deflection which occurred within the base also decreased. The authors suggested that this was due to the greater confinement in the base which resulted from an increase in the ratio of the load diameter to base thickness. This stress dependency was also indicated in field tests by Brown and Pell.[38] Using plate load tests on a prototype pavement, the modulus of a well-graded crushed stone was determined from measurements of in situ stresses and strains. They found that the modulus could be represented by Equation 4.4 where K_1' and K_2' were equal to 2040 and 0.57, respectively. Poisson's ratio was found to vary as follows:

Sum of Principal Stresses, psi	Poisson's Ratio
5	0.42
45	0.25

Similar stress dependency has also been observed using vibratory techniques. Jones[39] computed the dynamic shear modulus of a 3-ft layer of fine sand from the resonant frequency of a plate on the surface subjected to alternating stresses superimposed on static stresses ranging from 3 to 10 psi and found that the dynamic shear modulus increased with the static pressure. An increase in the alternating stress, however, resulted in a decrease in the dynamic modulus. Lister and Jones[40] used vibratory techniques to define the modulus of each layer in a pavement composed of a bituminous concrete surface, granular base, and subbase overlying a silty clay subgrade (CBR of 7.1). They observed that the overall stiffness of the structure increased with increasing load and attributed this to the nonlinear behavior of the granular layers. Accurate measures of the modulus of the base and subbase layers were not obtained. This stiffening behavior was also reported by Gusfeldt and Dempwolff.[41] They observed a slight nonlinearity between the tire load and the vertical stress in the gravel base (bituminous concrete surfacing and gravel base overlying a fine sand subgrade). A considerable stiffening behavior was noted in the relation between the load and the horizontal strain in the bituminous layer.

In repeated plate load tests on a prototype pavement, Hicks[9] found that the resilient response of pavements with granular bases depends on factors such as plate pressure, plate diameter, bituminous concrete thickness, and degree of saturation, and that for a given stress level the resilient surface deflections increased with increasing plate pressure, increasing plate diameter, and decreasing bituminous concrete thickness. The deflection at the subgrade-base interface increased in the same manner. The horizontal strains in the base layer varied with plate pressure, plate diameter, bituminous concrete thickness, and degree of saturation in the manner described for the deflections. The strains increased rapidly at the lower applied stresses and more slowly at the higher stresses. Tests with larger plates resulted not only in higher strains but also in lesser degrees of nonlinearity. However, some of the trends were not very clear.

4.2.5.3 Comparison of Test Results for Variable and Constant Confining Pressures. All results and findings of laboratory repeated triaxial tests on resilient property of granular materials reported in this section are based on constant confining pressures; test results with variable confining pressure reported by Allen[19] is the only one of its kind at present. As stated by Allen,

> Despite the knowledge of the heavy influence of confining stresses on the resilient parameters, there have been no published results of tests where lateral stresses were varied simultaneously with the axial stress on such a time scale as to simulate transient wheel loadings. Since this condition is representative of the state of stress occurring in an actual pavement structure subjected to moving wheel loads, the purpose of this research was to simulate field conditions by means of repeated-load, variable-confining-pressure, triaxial tests in the laboratory.

Some salient features of Allen's work are summarized in the following paragraphs.

Comparison of test results of variable confining pressure (VCP) and constant confining pressure (CCP) for resilient modulus E_r are shown in Figures 4.12-4.14. Test results for the crushed stone and gravel materials indicate that the CCP test yields slightly higher

values of E_r throughout the range of θ values for the intermediate- and low-density specimens than does the VCP test. This observation was also true for the low-density blend specimen for values of $\theta >$ 15 psi. The difference in E_r values in each case was maximum for values of θ near 10 psi, the lower extreme for θ . At this point, the CCP test on the intermediate-density crushed stone specimen showed E_r to be approximately 50 percent greater than the VCP test data indicated. This difference diminished as θ increased, since the regression lines converged at higher θ values. However, the differences in E_r for the other specimens mentioned above were considerably less, on the order of 30 percent maximum. Similar results were obtained from the high-density crushed stone and gravel specimens. For the gravel, the CCP and VCP test regression lines intersect at a value of θ near 17 psi; so for most of the range of interest the CCP test results yield higher values of E_r . However, for the crushed stone specimen, the point of intersection was $\theta = 35$ psi. For two specimens, the high- and intermediate-density blend specimens, the VCP and CCP test data result in almost identical regression lines for E_r (Figure 4.14).

On the basis of the above discussion, it would appear that, in general, the CCP test data would tend to overestimate the resilient modulus compared to the VCP test. However, two observations should be made. First, this phenomenon was not observed for all specimens. Second, in the cases where it did occur, the magnitude of the difference in E_r was not constant because of the intersecting or convergent nature of the regression lines; therefore, the magnitude of the difference depends upon the value of θ for which the values of E_r are calculated. It follows that the differences in the results of the two types of test may or may not be significant as regards pavement response to load since the modulus throughout the granular layers is determined from the existing state of stress.

The stress-dependent nature of the resilient Poisson's ratio is illustrated in Figures 4.15 and 4.16. The best fit to the laboratory

data was obtained for all specimens by expressing ν_r as a function of σ_1/σ_3. Figure 4.15 shows this relationship for the VCP test data from the low-density gravel specimen. Of interest is the relatively flat slope of the curve through the range of σ_1/σ_3 of 2 to 7. This observation indicates that, since this range of stress ratios is typical of that found in pavement systems, pavement analyses based on a representative constant value of Poisson's ratio for the granular layers might be appropriate. The validity of this observation is strengthened by the fact that the VCP test results for all specimens yielded values of Poisson's ratio very close to those shown in Figure 4.15 for the same range of σ_1/σ_3. Figure 4.16 shows ν_r values from the CCP test on the same specimen as Figure 4.15. This figure, typical of the CCP results for all specimens, differs in two respects from the typical VCP results shown in Figure 4.15. First, the curve is concave downward throughout the range of σ_1/σ_3 of interest, as opposed to the upward concavity evident in Figure 4.15. Second, the values of ν_r are much higher (>0.50) throughout the same range of σ_1/σ_3 for the CCP results than for the VCP test data. This contrast in behavior may indicate that the CCP test conditions cause the specimen to undergo more volume change than the VCP test. This is further explained in the following paragraph.

It can be shown that the volumetric strain of a specimen $\Delta v/v$ is equal to the first invarient of the strain tensor $\varepsilon_1 + 2\varepsilon_3$ for the triaxial test specimen. Examination of the detailed test data showed that at almost all stress levels applied during the CCP test $\Delta v/v$ calculated from the sum of the principal strains would indicate that the specimen increased in volume. However, applying the same procedure to the VCP test results would show little, if any, volume increase. Accordingly, it is felt that the conditions of the CCP test are such that inordinate degrees of volume change are imposed on the specimen, thereby yielding results that erroneously overestimate Poisson's ratio. In summary, no particular difference in the resilient modulus was observed in the two types of tests. However, the difference in resilient Poisson's ratio in the two types of test was very significant.

A sensitivity analysis on the significance of VCP on pavement response was made by Allen.[19] It consisted of the finite element analysis of one typical flexible pavement section. Twenty-one combinations of base and subbase course materials were utilized. The sensitivity analysis showed the necessity of accounting for the state of stress in granular layers when computing pavement response to load. However, any difference in the predictive equations for moduli derived from the two test procedures (VCP and CCP) exerts only minimal influence on such indicators of pavement response as surface deflections. Therefore, the continued use of the CCP triaxial test as a means of characterizing granular materials is justified.

Many more researchers have investigated the resilient properties of granular materials though their works were not reported in this section. Interested readers may consult References 26 to 42.

4.2.6 PLASTIC PROPERTIES

In the modern design of flexible pavements using a mechanistic, structural approach, fatigue and rutting are the two most important failure mechanisms. Fatigue in bituminous pavement surfaces is caused by the repetitive application of wheel loads which induce fluctuating stresses and strains in the surface, while rutting is due to permanent deformations in the pavement resulting from a combination of consolidation and shear failures. While rutting and fatigue are two separate modes of distress, the rutting can contribute to fatigue failure of a pavement due to the tensile strains in the surfacing resulting from bending caused by rutting of the base and subgrade.

To prevent pavement failure due to rutting, it has been the general practice to specify proper compaction control in the field and the use of appropriate pavement thickness to reduce stress intensities in the subgrade. The CBR equation devised by the Corps of Engineers is a classic example of a method which determines an appropriate thickness of pavement to protect the subgrade soil from shear failure. The Corps also has a construction procedure specifying a strict compaction requirement of the base, subbase, and subgrade soils to minimize

the possibility of consolidation of each layer under the repetitive application of traffic loads.

No effort has been made to investigate the characteristics of plastic deformation of pavement materials or to predict rut depths of a pavement using rational methods until very recently. Research in these areas by Barksdale,[42] Allen,[19] Kalcheff,[2] and Brown[33] is discussed below.

4.2.6.1 Georgia Institute of Technology. Barksdale[42] was first to investigate the plastic deformation of a variety of granular materials tested in a repeated load triaxial cell and developed a method for estimating the rut depth in a flexible pavement. Table 4.4 summarizes the 10 different kinds of base materials Barksdale tested in the repeated load triaxial cell. The specimens were tested to an average of 100,000 load repetitions at constant confining pressures of 3.5 and 10 psi. The tests were performed using deviator stresses varying from approximately 1 to 6 times the confining pressure.

The relationship for a granite gneiss (Base 6) between the axial plastic strain occurring in the cylindrical specimens and the numbers of load applications for varying deviator stresses is shown in Figure 4.17. The plastic strain accumulates approximately logarithmically with the number of load repetitions. For very low deviator stresses, the rate of accumulation of plastic strain tends to decrease as the number of load repetitions increases. As the deviator stress increases, a critical value is reached beyond which the rate of strain development tends to increase with increasing numbers of load repetitions. Furthermore, after a relatively large number of load repetitions, the specimen may undergo an unexpected increase in the rate of plastic strain accumulation.

To study rutting in pavement systems in a rational manner, the results given in Figure 4.17 can be plotted as plastic stress-strain curves shown in Figure 4.18. These curves are analogous to the stress-strain curves obtained from a series of static tests performed at varying confining pressures. Similar plots were also obtained for the

other nine base materials. The plastic stress-strain curves exhibit a typical nonlinear response. At a given confining pressure for small values of deviator stress, plastic strain is almost proportional to the deviator stress. As the deviator stress becomes greater, the development of plastic strain increases at an increasing rate until the plastic strains become very large as the apparent yield stress of the material is reached. Elastic strain is also strongly dependent upon the confining pressure, undergoing a significant decrease as the confining pressure increases.

A summary comparison of the plastic stress-strain characteristics of the base course materials investigated is given in Figure 4.19 for a confining pressure of 10 psi. Although the average confining pressure in a typical pavement structure is probably less than 10 psi, the comparisons are shown for this value since these stress-strain curves were more well defined. All materials compared on this figure were compacted to 100 percent of AASHTO T-180 density or its equivalent, except the silty sand which was compacted to 100 percent of T-99 density.

The base materials exhibiting by far the largest plastic strains were Base 1, a fine silty sand base, and Base 2 which was a 40-60 soil-aggregate base. For deviator stress ratios greater than 2.5 the measured plastic strains in the silty sand were larger than those in the 40-60 soil-aggregate base. Base 3, which was another 40-60 soil-aggregate base, exhibited approximately one half the plastic strains occurring in the first 40-60 soil-aggregate base due apparently to slight differences in the soil properties. For deviator stress ratios greater than 2.5, the average plastic strains in this soil-aggregate base were, however, still almost twice those occurring in Bases 4 and 5 which had only approximately 20 percent soil. Figure 4.19 shows that both soil aggregate bases tested having nominal 20-80 blends had significantly better plastic strain characteristics than did the two 40-60 bases. The plastic strain characteristics of the graded aggregate bases tested in the as-compacted condition were thus found to vary from very poor to quite good depending apparently on the soil characteristics, the percent of soil used in the base, and the degree of saturation.

For deviator stress ratios greater than about 2.5, the 17-83 soil-aggregate base (Base 4) exhibited significantly more plastic strain than did the best performing crushed stone base (Base 6) which had 3 percent fines. For deviator stress ratios less than 5, the plastic strains occurring in the 21-79 soil-aggregate base (Base 5) were on the average about 20 percent less than those occurring in the best crushed stone; at greater stress ratios, however, apparently the trend was reversed.

The curves shown on Figure 4.19 for the crushed stone bases indicate that the plastic strain occurring in the biotite granite gneiss (Bases 8 and 9) are greater than those in a porphyritic granite gneiss (Bases 6 and 7) for the same specified gradations. The significant influence of an increase in percent fines and deviator stress on the plastic strains occurring in a crushed biotite granite gneiss is illustrated in Figure 4.20. The plastic strains increased significantly as the percent fines increased, with greater differences occurring at the larger deviator stress levels.

A limited number of repeated load triaxial tests were performed on specimens at 90, 95, and 105 percent of maximum density. The results indicated that for all of the materials studied, an average of 185 percent increase in plastic strain occurs if the base is compacted at 95 instead of 100 percent of maximum compaction density. For an increase from 100 to 105 percent of maximum density, the corresponding average reduction in plastic strain was only about 10 percent, although more extensive testing may show the effect to be somewhat greater.

The experimental results also indicate that for all of the materials tested an average increase in plastic strain of 68 percent occurs when the test is performed on specimens that are soaked, as compared with the results obtained from tests performed on specimens in the as-compacted condition. It should be remembered that the soaked specimens had a high degree of saturation but may not have been completely saturated. These specimens were tested in a manner which permitted a free flow of water into and out of the specimen so that a significant

buildup of pore pressure was not likely to have occurred during application of the 100,000 load repetitions. Therefore, if a significant buildup of pore pressure should occur in the field in any of these materials due to poor drainage conditions, the laboratory test results would probably underpredict the effects that soaking of the base would have on the actual amount of rutting. Materials having the lower permeabilities such as the silty sand and graded aggregate bases would be more susceptible to such a pore pressure buildup in the field.

In summary, the plastic strains of granular materials increase with increasing deviator stress, decrease with increasing confining pressures, and increase significantly with increasing fines, with greater differences occurring at the larger deviator stress levels. Laboratory tests also revealed that the plastic strains increased drastically if the base is compacted at 95 instead of 100 percent of maximum compaction density. The failure of plastic deformation could be more serious than predicted in the laboratory if a significant buildup of pore pressures should occur in the field due to poor drainage conditions.

In order that the rutting characteristics of base materials can be easily compared, Barksdale[42] proposed the concept of a rut index. This is defined as the sum of the plastic strains in the center of the top and bottom halves of the base multiplied by 10^4. This requires that deviator and confining stresses be selected for a typical pavement and that plastic strains be obtained at a particular number of load repetitions. Barksdale presents rut indices at 10^5 load repetitions for the materials tested. These are given a numerical comparison representative of the curves shown in Figure 4.19 and are given in Table 4.5.

It is desirable to be able to predict rutting at higher numbers of load repetitions (i.e., 10^6 or more), but to test specimens to this extent would be time-consuming and expensive in practice. It is thought acceptable to extrapolate through one decade on a plastic strain versus log number of repetitions plot and then use the results to construct

plastic stress-strain curves from which the rut potential can be estimated. Rut potentials at 10^6 load repetitions are presented in Table 4.5.

The rut index and rut potential offer a rapid approximate comparison of rutting characteristics of base materials of the same thicknesses subject to the same loading and environmental conditions.

4.2.6.2 University of Illinois. Allen[19] conducted a series of laboratory repeated load triaxial tests on three different granular materials subjected to both constant and variable confining pressures. Test data indicated that the nonrecoverable deformations associated with the constant confining pressure (CCP) portion of the tests excluded those associated with the variable confining pressure (VCP) portion for every specimen. Table 4.6 shows the total plastic axial strain accumulated by each specimen during the entire test series. It also shows the percentage of the total plastic strain accrued during the CCP and VCP portions of each test series. From Table 4.6 it can be seen that the CCP portion of each test series resulted in from 2 percent to 56 percent greater plastic axial strains than the VCP test, although for most specimens the difference was around 8 to 10 percent. Finn[43] has shown that, on the basis of the Mohr-Coulomb yield criteria for soils, plastic strain is accompanied by volume change. From this viewpoint, the greater volume change observed during the CCP test is compatible with the greater resultant plastic strains.

4.2.6.3 National Crushed Stone Association. Extensive laboratory repeated load triaxial tests were conducted at the National Crushed Stone Association (NCSA) to study the characteristics of plastic deformation of graded aggregates. Kalcheff[2] reported that the plastic strains are greatly dependent on the degree of consolidation for the same gradation, the amount and type of fines in the gradation, the stress sequence and magnitude, and for some types of fimes the moisture content. The procedure is extremely useful for optimizing materials combinations or for the relative ranking of different materials at the same stress conditions. The NCSA investigations show that graded aggregates can be proportioned for minimum plastic deformations to provide a base that

improves its resistance to rutting with time and one which will not crack or lose stability with age. Figure 4.21 is an illustration of how density affects the plastic strains. The load magnitude for this material was the same.

Kalcheff[2] also illustrated in Figure 4.22 the effects of different types of fines on the plastic response of two types of aggregates. The gravel mix shown in the figure with either type of dust had the same elastic properties. Kalcheff thus emphasized that all graded aggregates do not have the same plastic strain responses under the same loading conditions even though their elastic properties and the quantity of fines may be the same.

Similar to the report made by Barksdale,[42] Kalcheff also noted the plastic strain accumulated approximately logarithmically with the number of load repetitions. In practice, the magnitude of plastic strain which may occur during the first year would double only after about 10 years carrying the same type of traffic. Kalcheff promoted the idea of stage construction which will provide time for the majority of plastic strains to take place when good clean stone base is used.

4.2.6.4 University of Nottingham. Brown[33] conducted a series of laboratory repeated load triaxial tests on a crushed granite of 5-mm maximum particle size. Plastic deformations were measured for each specimen. He found that under drained conditions the permanent strain reaches equilibrium values after approximately 10^4 cycles of deviator stress. The permanent strain at equilibrium could be related to the applied stresses by the equation $\varepsilon_p = 0.01(q/\sigma_3)$, where q is the effective deviator stress and σ_3 the confining pressure.

A recent extension of this work at the University of Nottingham[44] has investigated the influence of loading sequence and that of applying cyclic cell pressure to the same granular material. The limited study of loading sequence showed that the resilient modulus was unaffected by this but that permanent strain was significantly affected. The permanent strain which built up after successive applications of about 10^5 cycles of gradually increasing level was less than half the value resulting from the application of the highest stress level immediately.

This finding is similar to that reported by Monismith, Ogawa, and Freeme[45] for fine-grained soil.

4.2.7 VALIDITY OF SUPERPOSITION PRINCIPLE

Although pavement materials are known to be highly nonlinear (i.e., a nonlinear relation exists between the loads and the effect they produce), field studies conducted at WES have revealed that the principle of superposition is approximately valid, indicating linear theory is not unreasonable in application to pavement analysis. The results are explained briefly in the following paragraphs, and the details can be found in References 45 and 46.

In the homogeneous sand and clay test sections constructed at WES, stresses and deflections were measured at different locations under single and dual plates of various sizes and load intensities. The superposition principle was used on the measurements of single plates to develop the stress and deflection basins produced by dual plates, and then compare the results with the actual measurements under the dual plates. It was found that the comparisons agreed well, with better agreements observed in stress than in displacement and in clay than in sand. The principle of superposition was also applied to instrumentation data obtained in test item 3 of the multiple-wheel heavy gear load test sections.[46] The item consisted of a 3-in. bituminous concrete surface, a 6-in. graded crushed stone base, a 24-in. gravelly sand subbase, and a 4-CBR heavy clay subgrade soil. WES pressure cells, LVDT's, and other instruments were embedded in the pavement at different depths up to 12 ft. Stresses and deflections were measured for loadings by a C-5A 12-wheel gear assembly (360 kips), one twin-tandem component of a Boeing 747 dual-wheel assembly (120 kips) and many single-wheel loads. The results strongly indicate that the principle of superposition is reasonably valid for flexible pavements except at depths near the surface.

The analysis presented in References 45 and 46 demonstrates that the principle of superposition as applied to pavement design is approximately valid. The stresses and deflections under multiple-wheel loads

can be obtained from the correct stress and deflection basins of single-wheel loads by the use of the superposition principle. The implication is that the stresses and strains in the pavement, except near the surface, are so small that materials are stressed within or near their linear ranges; hence, linear analysis may not be the most critical factor in the disagreement between measurements and predictions. Evidently, there is a strong contradiction between prototype field measurements and laboratory findings in material behaviors. However, a satisfactory answer is not available at the present time.

4.3 CONSTITUTIVE STRESS-STRAIN RELATIONS

4.3.1 RESILIENT STRESS-STRAIN

The concept of resilient response of pavement materials was essentially developed at the University of California at Berkeley. Experience has shown that heavy-duty bituminous concrete pavements subjected to many repetitions of heavy load may develop cracks resulting from fatigue of the bituminous concrete. Available information indicates that the deflections inducing this form of pavement cracking are essentially transient. That is, the deformations in the pavement structure caused by the moving vehicles are essentially elastic in the sense that they are almost completely recoverable. To avoid confusion, however, the deflections have been termed resilient rather than elastic. Because these deflections are recoverable, it would appear that elastic theories of stress distribution can be used to represent the behavior of pavements from a theoretical standpoint.

The resilient modulus is defined to be the ratio of the repeated axial stress in triaxial compression divided by the recoverable axial strain. The same concept is used for resilient Poisson's ratio. Basically, this approach seeks to formulate predictive equations for the resilient modulus and resilient Poisson's ratio through the use of repeated load triaxial tests. By expressing these parameters as functions of the state of stress in the materials, it is possible to account

for nonlinear material response. The derived modulus and Poisson's ratio may then be used to characterize the pavement materials in the numerical solution for transient pavement responses; i.e., stresses, strains, and deflections.

Many academic institutions devised resilient stress-strain relations for granular materials based on laboratory test results. On the other hand, many nonacademic institutions developed empirical stress-strain relations based on their own experiences and environmental conditions. They are discussed separately below.

4.3.1.1 University of California at Berkeley. Two models have been used to describe the stress-dependent resilient modulus of granular materials

$$M_R = K_1 \sigma_3^{K_2} \quad (4.3 \text{ bis})$$

and

$$M_R = K_1' \theta^{K_2'} \quad (4.4 \text{ bis})$$

where

K_1, K_2, K_1', and K_2' = material constants

σ_3 = confining pressure

θ = sum of principal stresses $\sigma_1 + 2\sigma_3$

Tables 4.7 and 4.8 summarize the material constants determined in the laboratory triaxial tests for various granular materials. For completeness, the constants determined by Dunlap[12] in Equation 4.2 are also included. Table 4.7 is taken from Barker, Brabston, and Townsend,[47] and Table 4.8 is taken from Barksdale and Hicks.[48]

Equations 4.3 and 4.4 are capable of characterizing granular materials in a nonlinear fashion; i.e., M_R varies with the state of stress, provided shear failure does not occur. However, these equations are incapable of characterizing tensile properties of granular materials. In a conventional flexible pavement under heavy loads, it is conceivable that radial tensile stresses would develop at the lower part of the

granular base. For most computer programs available at present, an arbitrarily small value of M_R close to zero is assigned to the granular material once M_R computed from Equations 4.3 and 4.4 becomes negative. Under such cases, awkward values of strain components and excessively large displacements result. In this respect, Equation 4.4 is far superior to Equation 4.3; this is explained in the following paragraph.

In Equation 4.4, θ is the sum of principal stresses σ_1, σ_2, and σ_3. When radial tensile stresses are developed at the lower portion of the base, the minor principal stress σ_3 most likely becomes negative and causes M_R to become negative in Equation 4.4. But, since the major principal stress σ_1 is positive and is much larger than σ_3 under most stress levels, M_R computed by Equation 4.4 is positive in most cases. Therefore, the moduli M_R computed by Equation 4.4 are more meaningful in many cases.

The relationship between resilient Poisson's ratio and the state of stress is written as follows based on test results:

$$\nu = A_o + A_1\left(\frac{\sigma_1}{\sigma_3}\right) + A_2\left(\frac{\sigma_1}{\sigma_3}\right)^2 + A_3\left(\frac{\sigma_1}{\sigma_3}\right)^3 \tag{4.6}$$

where the A constants are determined from regression analyses. However, because of lack of test data, resilient Poisson's ratio values were chosen arbitrarily in most computations.

4.3.1.2 University of Illinois. A series of laboratory repeated load triaxial tests on three different granular materials was conducted at the University of Illinois.[19] The triaxial tests were conducted at both constant confining pressure (CCP) and variable confining pressure (VCP) conditions. Table 4.9 describes the test specimens. Equations 4.3, 4.4, and 4.6 were also used to model the resilient modulus and Poisson's ratio. Tables 4.10 and 4.11 show the equations for resilient modulus and Poisson's ratio, respectively, for each specimen formulated by regression equations from plotted test data. As stated earlier, the difference in resilient modulus between VCP and CCP tests was not very

significant, but the difference in resilient Poisson's ratio between these two tests was significant.

4.3.1.3 University of Nottingham, England. A series of laboratory repeated load triaxial tests with constant confining pressure on a crushed granite were conducted by Brown.[33] The expression for resilient modulus in terms of principal stresses was derived from the particular aggregate

$$M_R = 91q\left(1 + \frac{\theta}{q}\right)^{1.54} \tag{4.7}$$

Where q and θ are effective deviator stress and mean normal stress.

4.3.1.4 Shell Procedure. In 1965, Shell Oil Company[49] first published its procedure for assigning modulus values to unbound granular materials. The procedure was determined arbitrarily by correlating with the CBR performance equation developed by the Corps of Engineers.[50]

In the procedure, no distinction is made between base and subbase materials, and the average modulus of the entire granular layer is determined based on the thickness of the layer and the modulus of the supporting subgrade. It is also independent of the magnitude of the load and the modulus and thickness of the overlying bituminous concrete layer. Figure 4.23 shows the relationship between thickness and modulus ratio developed by the Shell Oil Company. Because of lack of a better and simpler procedure, the Shell procedure has been adopted widely by engineers for pavement design and analysis. Recently, Chou[51] made a review of the applicability of the Shell procedure based on the results of full-scale accelerated traffic tests conducted at WES. It was shown that a distinct difference in performance exists between granular materials of different qualities. Moduli determined using the Shell procedure seem to be reasonable for subbase and low-quality base materials. However, for pavements with thick, high-quality, well-graded crushed stone bases, moduli higher than those determined using the Shell procedure should be considered.

4.3.1.5 Kentucky Highway Department. Similar to the Shell procedure,[49] the procedure devised by the Kentucky Highway Department[52]

for assigning modulus values to granular layers is based on the modulus of the underlying subgrade soil. According to the department's many years of field experience, the range of the ratio is generally between 1.5 and 4. A value of 2.8 was selected as being typical at a CBR of 7. The relation is shown in Figure 4.24. The notable features of this procedure are that no distinction is made between base and subbase materials and the modulus is independent of the layer thickness, the magnitude of the load, and the thickness of the overlying bituminous concrete layer but dependent on the modulus of the overlying bituminous concrete.

Since the variations in the types of aggregates, thicknesses of granular layer and bituminous concrete surface, and loading conditions may not be large for the flexible pavements designed in the State of Kentucky, the procedure for assigning modulus values to granular layers, which is dependent only on the modulus of the bituminous concrete surface (or temperature) and the modulus of underlying subgrade soil and not dependent on other factors mentioned in the last paragraph, seems to be reasonable.

4.3.1.6 U. S. Army Engineer Waterways Experiment Station. In the current design procedure for airfield flexible pavements,[53] the design CBR for select materials and subbases supplemented by gradation and Atterberg limits and the assigned CBR for bases are shown in Table 4.12. Attempts were not made to use the CBR values to compute the stresses and strains, rather, the CBR values were used to determine the required thickness to protect the supporting soil from shear failure.

Considerable effort to establish failure criteria for flexible pavements based on observed full-scale accelerated traffic test data has been made at WES in recent years. Computer programs were used to calculate stress and strain in the test pavements. Because of lack of laboratory test results characterizing the material properties used in the test pavements, the values of elastic modulus were determined by some means that appeared to be sound and logical. Poisson's ratios were selected in an arbitrary manner.

In References 46 and 54, a structural analysis of flexible airfield pavements is described; maximum shearing strains at the top of subgrade soil were computed using Burmister's linear layered elastic theory. The elastic moduli of granular layers were evaluated based on the theory behind the CBR equation; i.e., a pavement designed by the CBR equation has a thickness sufficient to prevent shear failure in the subgrade soil. Consequently, two pavements designed by the CBR equation for the same coverage level can be expected to experience approximately equal shearing strain at the surface of the subgrade soils. Based on this principle, the elastic moduli of various pavement materials of conventional airfield flexible pavements were evaluated.

For subgrade soils of different CBR values, pavement thicknesses designed at capacity operations (5000 coverages) for different single-wheel loads were computed by using the CBR equation[50]

$$t = \sqrt{\frac{P}{8.1\ \mathrm{CBR}} - \frac{A}{\pi}} \tag{4.8}$$

where P is the wheel load and A is the contact area.

By considering the total pavement thickness as a single layer (i.e., neglecting the differences in the structural rigidity of the surface, base, and subbase materials) the maximum shearing strains were computed at the surface of the subgrade soil for different CBR ratios. For instance, for a 4-CBR subgrade soil, four different computations were carried out with the CBR values of the pavement structure above the subgrade being 8, 20, 32, and 40, which corresponded to ratios of 2, 5, 8, and 10, respectively. The computations were made by the Chevron program,[55] which is based on Burmister's layered elastic theory. To convert CBR into elastic modulus E for use in the computer program, the relation $E = 1500\ \mathrm{CBR}$ was used.

The computed results are shown in Figure 4.25. It can be seen that the computed values all plot along a smooth curve, indicating that pavements designed by the CBR equation yield equal computed maximum shearing strains at the surface of the subgrade soil when the computations are carried out for selected modulus ratios. With a ratio of 3,

for instance, the maximum shearing strain γ_{max} at the surface of a 4-CBR subgrade soil subject to a 10-kip load is equal to γ_{max} at the surface of a 20-CBR subgrade soil subject to a 30-kip load (if 12 and 60 CBR, respectively, are assigned to the layers above the subgrade in the computations for these two pavements). Similarly, when the same loadings and a ratio of 5 are used, the maximum shearing strains would also be equal if 20 and 100 CBR were assigned respectively to layers above the subgrade. The same principle is applicable to other ratios. It should be pointed out here that the shearing strains so obtained are merely computed values, and their physical meanings are not specified.

The differences in the structural rigidities of component layers are considered in the layered analysis. The thickness of bituminous concrete and base course (well-compacted crushed stone) under different wheel loads were determined using the Corps of Engineers standard flexible pavement design procedure. For all the pavements designed, the minimum thickness of subbase layer (sandy gravel) was 4 in.

In computations, the CBR value of the bituminous concrete was assumed to increase with increasing thickness of the layer because of its temperature-dependent nature. The thickness-CBR (or elastic modulus) relation used follows that developed by the Shell group,[56] in which the CBR is exactly 100 at a thickness of 3 in. The CBR value of the base course material was assumed to be 1.5 times greater than the subbase material. In the case of strong subgrade soils when the CBR value of the base course exceeded that of the bituminous concrete layer, the latter was arbitrarily increased to be equal to the former.

Figure 4.26 shows the result of layered analysis plotted in a manner similar to Figure 4.25. For a given wheel load, computations were carried out for several pavements with subgrade soils of different strengths. It was found that computed values could all be plotted along a smooth curve with almost no scattering. Each curve shown in Figure 4.26 represents the average of computed values for several pavements.

The results shown in Figure 4.26 indicate some very significant facts about the behavior of flexible pavements under aircraft loads.

First, the thickness of the granular layer has no significant effect on its effective elastic modulus (or CBR value). Second, the effective modulus of a granular material is not a constant but depends on the modulus of underlying materials. Also, equal maximum shearing strains at the surface of subgrade soils can be obtained for two pavements designed by the CBR equation if the computations are carried out for the same modulus ratio, such as 2.8 used by the Kentucky Department of Highways.[52] To further illustrate this point, the maximum shearing strains at the top of subgrade soil of many test pavements failed by full-scale accelerated traffics were computed. The computed values were plotted against the observed performance data. They are shown in Figure 4.27. The computations were carried out for three different CBR ratios. It is apparent that good correlations can always be obtained when computations are based on selected modulus ratios. This fact lends support to the conclusion previously drawn in connection with Figures 4.25 and 4.26.

In a study presented at the Transportation Research Board Meeting in 1974,[57] the nonlinear finite element technique was employed to analyze many full-scale test pavements which were failed by accelerated traffic of both single- and multiple-wheel loads. Strains at critical locations in the test pavements were computed and were correlated with the observed performance of the test pavements. The correlations were very good. In the computations, the stress-dependent moduli of granular base and subbase materials were computed from the equation

$$M_R = K_1' \theta^{K_2'} \qquad (4.4 \text{ bis})$$

where θ is the first stress invarient and K_1' and K_2' are constants describing the nonlinear characteristics of the materials. K_1' and K_2' values were determined by a trial and error procedure in such a manner that best correlations between computed strains and actual performance were obtained. K_1' and K_2' were determined using this procedure as 8300 and 0.71 for a high-quality, well-graded crushed stone base material and as 2900 and 0.47 for sand and gravel subbase material.

To check the accuracy of K_1' and K_2' values in Equation 4.4 for granular materials, the stresses and deflections computed by the nonlinear finite element method were compared with those measured (under static loading) in a full-scale test pavement[58] constructed and tested at WES. The test pavement consisted of a 3-in. bituminous concrete layer, a 6-in. high-quality, well-graded crushed stone base, and a 24-in. gravelly sand subbase on a 4-CBR subgrade. Vertical stresses and deflections were measured using LVDT's and WES 6-in. pressure cells installed at various depths in the pavement. The details of the design and construction of the test pavements and of the procedures of measuring responses are described in Reference 58. In the computations, a constant modulus of 100,000 psi was used. This value is associated with a pavement temperature of 90° F during the time of measurement. The resilient modulus of the subgrade soil was determined in the laboratory from undisturbed samples.

Figure 4.28 shows comparisons between computed and measured elastic (or rebound) deflections along the load axis of 15- and 50-kip single-wheel loads. Figure 4.29 shows the vertical stresses for these two single-wheel loads. It can be seen that the computed deflections are in good agreement with the measured ones, indicating that the resilient moduli determined for the granular materials are of the correct magnitudes. However, it should be pointed out that, since the test pavement had a thin base layer and the response of the pavement to the loads was affected only slightly by the base layer, the agreement between the computations and the measurements does not necessarily guarantee that the modulus used for the base layer was correct. The agreement does indicate, however, that the moduli for the subbase and subgrade were correct.

The nonlinear stress-strain relations for granular materials developed by Chou[57] compare favorably with those developed by The Asphalt Institute.[59] The comparisons are presented later in this chapter.

In the development of a design procedure for all-bituminous concrete pavements for military roads,[60] the Chevron program was used

to compute vertical strains at the top of subgrade soil of many hypothetical pavements. For computation purposes, the granular base and subbase layers were divided into many layers, each approximately 6 to 8 in. thick. The modulus of each layer was estimated based on a modulus ratio concept. The ratio relationships, from an analysis of previous pavement tests, are shown in Figure 4.30. These relationships were used to estimate the modulus value of the sublayers or layers in ascending order by multiplying the ratio determined by the thickness of the layer under consideration by the modulus value of the underlying layer. A relationship, similar to that in Figure 4.30, developed by Barker and Brabston[61] is shown in Figure 4.31. The use of Figure 4.31 is essentially the same as Figure 4.30, except that Figure 4.31 is in descending order.

4.3.1.7 Asphalt Institute. Witczak[59] developed a design procedure for full-depth bituminous concrete airfield pavements. For the permanent deformation mode of distress, he established the failure criteria based on the Corps of Engineers traffic test data.[58] For the convenience of discussion, the procedure is briefly described in the following paragraphs.

The total thicknesses of a number of hypothetical pavements were designed using the Corps of Engineers design procedure for multiple-wheel loads at various coverage levels.[58] With a bituminous concrete surface of 5 in., the maximum vertical strains at the top of subgrade soil were computed for a variety of aircraft using the BISTRO computer program. The computations were made based upon the following assumptions: (a) no distinction was made between base and subbase materials; (b) Poisson's ratios of 0.40 for bituminous concrete and 0.45 for unbound layers were used; (c) the subgrade modulus E_s was equal to 1500 CBR; (d) the pass (operation) per coverage factor for channelized taxiway traffic conditions established by the Corps of Engineers was valid;[61] (e) 1 coverage equalled 1 strain repetition; and (f) the modulus ratio of the unbound granular layer to the subgrade was variable and dependent upon the modulus of the subgrade. Approximate ratios used were 2.9, 2.3, and 1.8 for CBR values of 3, 5, and 10, respectively. These values were selected

from a theoretical study of the stress dependence of the resilient modulus of the granular layers, as evaluated from laboratory tests. Witczak pointed out that ratios so obtained are in excellent agreement with dynamic modulus relationships developed from in situ field studies.[62] Results of the analysis indicate that, for all practical purposes, the effect of the aircraft type and subgrade modulus values investigated yielded insignificant differences in the maximum vertical strain-repetition relationship.

Figure 4.32 illustrates the vertical subgrade strain-repetition relationship for three different aircraft evaluated at a bituminous concrete stiffness of 100,000 psi. Figure 4.33 shows maximum vertical subgrade strains for various bituminous concrete moduli E_1 determined from a strain analysis of a 3-layer pavement system. It can be seen that the vertical subgrade strain decreases as the surface modulus E_1 increases. As pointed out by Witczak,[59] the curves shown in Figure 4.34 assume that the increase in stiffness of the 5-in. wearing surface results in no increase of the allowable number of repetitions to failure, and are therefore a conservative approach. Further explanation of this is given in the next paragraph.

The stiffness of the wearing surface is not a required input in the use of the Corps of Engineers design procedure. Since data from traffic tests conducted at WES were collected essentially at a rather warm climatic condition, an E_1 value of 100,000 psi for bituminous concrete as used in Figure 4.32 appears to be a representative value, although it may be slightly too high. For a given test pavement, when the stiffness of the bituminous concrete is increased (or the temperature decreases), it is conceivable that the performance of the test pavement would become better as far as the subgrade shear failure mode of distress is concerned. Therefore, the relationship expressed in Figure 4.33 is pindeed conservative and would not exist in reality for traffic tests conducted under colder climatic conditions (i.e., with higher E_1 values); curves 2-5 in Figure 4.33 would approach very closely curve 1. In other words, the failure criterion shown in Figure 4.32 should be applicable, for practical design purpose, to any temperature conditions. This may

become clear when Figure 4.32 is compared with the failure criterion developed at WES, which was based on results of numerous full-scale traffic tests (essentially the same used by Witczak) incorporated with the nonlinear finite element analysis. The comparison is shown in Figure 4.34. The detailed development of the WES failure criterion can be found in Reference 57 (Figure 5). It can be seen that the comparison is excellent, although Witczak's computations were carried out using the linear BISTRO program and Chou's computations were carried out using the nonlinear finite element program. A study was also carried out to investigate the stress-strain relations proposed by Chou[57] and Witczak.[59] The results are presented in the following paragraphs.

Thirteen pavements were designed at different coverage levels based on the CBR equation,[51] which is

$$t = (0.15 + 0.23 \log C)\sqrt{\frac{P}{8.1\ \mathrm{CBR}} - \frac{A}{\pi}} \tag{4.9}$$

where P is the wheel load, A is the tire contact area, and C is the number of coverages. (When C is equal to 5000, the expression $(0.15 + 0.23 \log C)$ becomes unity.) The required thicknesses of bituminous concrete and base course of the test pavements under different wheel loads were determined by the Corps of Engineers standard flexible pavement design procedure. The pavements are described in Table 4.13.

Two series of computations were made to compute the vertical subgrade strains of the 13 pavements. The nonlinear finite element program incorporated with Equation 4.4 characterizing the nonlinear properties of granular layers, with K_1' and K_2' equal to 8300 and 0.71 for base course and 2900 and 0.47 for subbase course, as proposed by Chou,[57] was used in one series. The linear BISTRO program with constant modulus ratios proposed by Witczak[59] was used in the other series. In both cases, a constant modulus of 41,000 psi was assigned to the bituminous concrete layer and the relation $E = 1500$ CBR was used for the subgrade soil. The computed strains versus the corresponding failure coverages are plotted in Figure 4.35 for the two series of computations.

It can be seen that the computed results are very close, indicating that the stress-strain relations for unbound granular materials developed by The Asphalt Institute[59] and WES[57] agree. The small difference at the low coverage levels in Figure 4.35 may be due to the assumption made by The Asphalt Institute[58] that the base and subbase have the same modulus values. At low coverage levels, the pavements are generally thin (see Table 4.13) and the effect of the 6-in. base layer is more significant as compared with pavements designed for higher coverage levels in which the thicker subbase layer dominates the pavement performance. (This is true only for subgrade failure.) With the base modulus underestimated, the computed subgrade strains at low coverage levels become larger than they actually should be.

4.3.2 PLASTIC STRESS-STRAIN

Barksdale[42] conducted a series of laboratory repeated load triaxial tests on a variety of base materials. The specimens were tested to an average of 100,000 load repetitions at constant confining pressures. Permanent axial deformations occurring throughout the entire height of the specimens were measured. Figures 4.18-4.20 show the patterns of the plastic stress-strain curves.

Following the hyperbolic expression developed by Duncan and Chang,[63]

$$\varepsilon_a = \frac{(\sigma_1 - \sigma_3)/(K\sigma_3^n)}{1 - \left[\dfrac{(\sigma_1 - \sigma_3)R_f}{\dfrac{2(C\cos\phi + \sigma_3\sin\phi)}{1 - \sin\phi}}\right]} \tag{4.10}$$

where

ε_a = axial strain

$K\sigma_3^n$ = relationship defining the initial tangent modulus as a function of confining pressure σ_3 (K and n are constants)

R_f = constant relating compressive strength to an asymptotic stress difference

C = cohesion

ϕ = angle of internal friction

Barksdale found that Equation 4.10 can fit the plastic stress-strain curves obtained from the repeated load triaxial test results for 100,000 load repetitions. An example of a hyperbolic fit of the plastic stress-strain curves obtained from the repeated load triaxial test is shown in Figure 4.36. The theoretical curves are almost identical to the laboratory curves at confining pressures of 3 and 5 psi; at a confining pressure of 10 psi, the calculated plastic strains are slightly greater than the laboratory values. The constants required in Equation 4.10 to calculate the curves are given in the figure.

Brown[33] conducted a series of laboratory repeated triaxial tests on a crushed granite of 5-mm maximum particle size. The permanent strain at equilibrium was related to the applied stresses by the equation $E_p = 0.01(q/\sigma_3)$, where q is the effective deviator stress.

4.3.3 SHEAR STRESS-STRAIN

The development of shear stress-strain relations of pavement materials (i.e., base and subgrade soils) has been carried out at the University of Kentucky since late 1960. The research is part of an Air Force Weapons Laboratory project to develop a pavement evaluation procedure. To facilitate the presentation, the concept involved in the pavement evaluation procedure is described briefly as follows:

a. Step 1. Measure the shear modulus for different layers of the pavement structure in situ, using a nondestructive vibration testing method.

b. Step 2. Make the proper reduction in the measured shear modulus to correspond to the strain produced by an aircraft loading.

c. Step 3. Use this shear modulus in a linear finite element analysis for stresses and strains in the pavement structure under load.

d. Final Step. Assess the amount of damage or pavement distress that will be produced by the loading.

The primary work involved at the University of Kentucky is to develop expressions for shear modulus and Poisson's ratio for various soils at different strain levels. Soil specimens were tested in the simple shear device. The research is in progress and the results are

reported in References 64-66. Some background materials may be found in References 67-68.

The shear modulus G is given by the expression

$$\frac{G}{G_{max}} = \frac{1}{1 + \gamma_h} \tag{4.11}$$

where

G_{max} = maximum shear modulus, which is the initial tangent modulus or recent modulus for strain amplitude $\leq 10^{-5}$ in./in.

For pavement evaluation, this quantity is to be measured by the nondestructive vibratory test

$$\gamma_h = \frac{\gamma}{\gamma_r}\left[1 + a e_{xp}^{-\left(\frac{\gamma}{\gamma_r}\right)^{0.4}}\right] \tag{4.12}$$

and is called hyperbolic strain

where

γ = shear strain

γ_r = reference strain is empirically related to G_{max}

The value of a is defined by one of the following equations, depending on the type of soil:

$$a = \left\{\begin{array}{ll} (3.85/N) - 0.85T^{0.025} & \text{for clean dry sands} \\ 1.6(1 + 0.02S)T^{0.2}/N^{0.6} & \text{for nonplastic soils with fines and low plasticity soils} \\ 0.2(1 + 0.02S)T^{0.75}/N^{0.15} & \text{for high plasticity soils with liquid limit} > 50 \end{array}\right\} \tag{4.13}$$

where

N = number of cycles

T = strain time, in the time in minutes to reach a normalized strain equal to one, where the normalized strain is defined to be γ/γ_r

S = percent saturation

In Reference 65 examples are given to illustrate the procedures to determine the shear modulus G for different soils. The required input information is the following:

a. G_{max} measured by the nondestructive vibratory testing.

b. Void ratio e .

c. Percent saturation S .

d. N determined from traffic record.

e. γ determined from the finite element analysis.

f. T dependent on the speed of the aircraft. For T values of 81 and 0.38, they are considered to be a very slow rate and a medium fast rate of landing.

g. Plasticity index and percent passing No. 200 sieve (needed only for fine-grained soil).

Using charts and figures in Reference 65, the shear modulus of the soil can be readily determined.

4.3.4 DYNAMIC STRESS-STRAIN

4.3.4.1 Modulus and CBR Relations. Vibratory testing was begun as early as 1933 by the German Research Society for Soil Mechanics and was further developed by the Royal Dutch Shell Laboratory and the Transport and Road Research Laboratory. WES commenced vibratory testing of pavement systems in search of nondestructive evaluation procedures in cooperation with the Shell researchers in the mid-1950's. These early tests by WES followed the procedures used by Shell researchers. The results of these early studies have been reported by Heukelom and Foster[69] and by Maxwell.[70,71] Based on an extensive field test, Heukelom[72] and Heukelom and Klomp[73] developed a correlation between E* modulus and CBR as

$$E \text{ (in psi)} = 1500 \text{ CBR}$$
$$E \text{ (in kg/cm}^2\text{)} = 110 \text{ CBR} \qquad (4.14)$$

The CBR range of data for this correlation is 50 to 200. Kirwan and Glynn[74] developed a similar relationship for two boulder clays

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* The procedure to determine E modulus is explained later in this section.

$$E \text{ (in psi)} = 250 \text{ CBR} \qquad (4.15)$$

The enormous variations in these correlations indicate the magnitude of error which can result from using such relationships. The scattering of data may result from the fact that the pavement materials under the small vibratory load are stresses only in the elastic range and the deformation is recoverable. On the other hand, the soils in the CBR tests are stressed to the plastic range and the deformations are not totally recoverable.

Table 4.14 contains the results of numerous measured dynamic modulus and CBR values from different sources. An arithmetic plot of these data is shown in Figure 4.37. The information is obtained from Reference 80. A linear regression analysis was performed to determine the best correlation equation

$$\log E = 3.73313 + 0.71145 \log \text{CBR} \qquad (4.16)$$

In recent years, a considerable effort has been made at WES to develop a nondestructive testing technique to determine the pertinent characteristics of pavement systems. The results are reported in References 81-91. Presently (as of 1975), there are three basic nondestructive testing techniques under study at WES. These are:

a. The use of steady state vibratory loadings and wave propagation measurements to determine the thicknesses and elastic constants of the pavement system, which can be used in a multilayered analysis to predict allowable loadings.

b. The use of steady state vibratory loadings and measurements of the resulting elastic deflections to determine a dynamic stiffness modulus (DSM) of the pavement system, which can be correlated with pavement performance.

c. The use of a theoretical approach based upon the amount of energy that a pavement system can absorb versus the amount of energy imparted to the pavement by aircraft traffic.

Procedures a and b have been the subject of considerable study, whereas procedure c is a more recent development.

In the full-scale multiple-wheel heavy gear load study conducted at WES during 1969-1970 to validate pavement design, vibratory equipment was used to monitor the performance of the pavement test sections. Tests to determine load-deflection relations (DSM values) and wave propagations were conducted periodically during this study. An analysis of the results showed that the DSM values correlated well with the performance data. However, the wave propagation results were erratic, with the computed modulus of elasticity E of the subgrade material varying apparently as a function of the overburden pressures exhibited by the different pavement thicknesses; the results seemed to be reasonable when the measurements were made on a uniform soil mass.

The procedures and equations used to determine the modulus and Poisson's ratio of soils using the vibratory testing technique are presented below. The discussion is from Reference 81.

4.3.4.2 Field Wave Velocity Measurements. Wave velocity measurements employ both the heavy mechanical vibrator of frequency range 6 to 8 Hz and the electrodynamic vibrator of frequency range 20 to 3000 Hz. A vibrator is placed on the pavement surface and a transducer is placed on the surface at various distances from the vibrator. By means of an appropriate phase-marking circuit and an oscilloscope, the length of the wave being propagated can be determined. This is done by locating a point on the surface where the phase marker coincides with the next corresponding peak (or trough). The distance measured on the surface between the points is 1 wavelength. By knowing the frequency, the velocity V can be determined from the simple relation:

$$V = f\lambda \tag{4.17}$$

where

f = frequency, Hz

λ = wavelength, ft

The process is repeated with other frequencies, and data are thus obtained to establish the relation of wavelength and velocity.

Detailed lists of equipment including several makes and models necessary for vibratory testing of soils and pavements are given in the literature (References 92-94). These references also give step-by-step procedures for field testing and data analysis.

4.3.4.3 Poisson's Ratio. Poisson's ratio ν can be determined by the relation of shear wave (or Rayleigh wave) velocities V_s and compression wave velocities V_c (Reference 92). The shear wave velocities are determined by vibratory tests mentioned above, and the compression wave velocities by ordinary seismic refraction measurements. The relation is

$$\nu = \frac{1 - 2\left(\frac{V_s}{V_c}\right)^2}{2 - 2\left(\frac{V_s}{V_c}\right)^2} \qquad (4.18)$$

Poisson's ratio of 0.5 is sometimes used to simplify calculations, although a value of about 0.4 is usually considered more nearly correct.

4.3.4.4 Modulus of Elasticity (Dynamic E Modulus). The shear modulus G of an elastic medium is related to the shear wave velocity as

$$G = \rho V^2 \qquad (4.19)$$

where

ρ = mass density = $\frac{\gamma}{g}$

γ = wet density of soil, pcf

g = acceleration due to gravity = 32.2 ft/sec^2

V = velocity, ft/sec

The shear modulus is then related to the compression modulus (E modulus) by

$$E = 2(1 + \nu)G \qquad (4.20)$$

and it can be derived that E is given by

$$E = 2(1 + \nu)\rho V^2$$
$$= 3\rho V^2 \tag{4.21}$$

for $\nu = 0.5$. A small variation in either ν or the unit weight of the material has little effect on the value of the E modulus (Reference 95).

4.4 PREDICTION OF PERMANENT DEFORMATION

Since Barksdale[42] proposed the procedure for estimating the rut depth in flexible pavement, a study was initiated at WES to analyze permanent deformations in flexible airport pavements subjected to moving aircraft loadings.[96] Pavements used in the analysis were full-scale multiple-wheel heavy gear load test sections constructed and tested at WES. Series of laboratory repeated load tests measuring permanent strain were performed on untreated granular materials and subgrade soils which were obtained from the test sections. The results were used as input to a layered elastic computer program to determine the accumulated permanent deformation that occurred in each layer of the pavement. Difficulty was encountered in the computations of permanent deformations in granular materials due to the fact that the radial stresses computed by the layered elastic program do not truly represent field conditions. The experience gained from the study is presented below.

a. When the layered elastic computer programs were used to obtain information on the stress states in the pavement structures, tensile radial stresses were generally computed at the bottom layers of the granular materials. This posed a serious problem in the use of laboratory repeated load test data.

b. In conducting laboratory repeated load tests on untreated granular materials, confining pressures were required during the test to prevent the specimen from collapsing under the load applications. The magnitude of the required confining pressure σ_3 depended upon the magnitude of the applied vertical pressure σ_1 . In general, the ratio of σ_1/σ_3 could not exceed a value of 5. In other words, if the applied vertical stress σ_1 was 20 psi, the confining stress σ_3 must be kept at 4 psi or greater.

When the stress states in the granular layers were computed and expressed by the relation $(\sigma_1 - \sigma_3)/\sigma_3$ (as shown in Figure 4.19), the expression becomes negative when tensile radial stresses σ_3 were computed. Consequently, the laboratory repeated load test data could not be used to estimate the permanent strains because the tests were conducted with compressive confining pressures; i.e., σ_3 was always positive. To circumvent the situation, the ratios of octahedral shear stress to octahedral normal stress τ_{oct}/σ_{oct} were used. The advantage of this expression is that the stress ratio was always positive even though the value of σ_3 was negative. Using the octahedral stress ratios, however, difficulties still existed, because extrapolations had to be used to estimate the permanent strains at high stress ratios.

The results of the study conducted at WES illustrated that the problem of estimating permanent deformations in granular layers using a mechanistic approach lies in the difficulty of computing the stress states. It is believed that when the wheel loads are applied on the pavement surface, radial *tensile stresses* start to develop at the lower part of the granular base layer and slip of the material becomes incipient. The granular material can sustain a certain amount of tensile stresses which are resisted by frictional stresses developed between the granular particles caused by the vertical compressive stresses that exist in the base. Once the material starts to slip, passive pressure due to overburden will be mobilized and the confining pressure will be decreased. Since energy is dissipated during the movement, the stress intensities may be substantially changed as compared to those during the stage of stress buildup. Since the magnitude of vertical compressive stress under a wheel load depends upon the magnitude of confining pressure in granular materials, the magnitude of vertical compressive stress σ_1 in a pavement structure may not be a constant but rather may vary during the loading process. Also, since aircraft loadings are not always applied at one point but vary laterally along the center line of the runway, it is likely that material in a pavement may move in directions other than the vertical when the load is not directly over the point where the material is located. It is obvious that (a) the states of stress

existing in the granular layers under aircraft loadings are extremely complicated, which cannot be simply described by constant values of vertical compressive stress σ_1 and horizontal stress σ_3, which are computed by the layered elastic program; and (b) the response of the granular materials to the repeated applications of aircraft loads cannot be simulated by the laboratory repeated load triaxial tests.

4.5 LABORATORY TESTING EQUIPMENT AND METHODS

Granular materials are nonhomogeneous, anisotropic, nonlinear, and nonelastic. Although they are not time- and temperature-dependent, they are affected to some degree by changes in moisture content. A detailed list of variables affecting general pavement material response reported by Deacon[97] is shown in Table 4.15.

4.5.1 TESTING EQUIPMENT

In order to determine the elastic property of granular materials for use in mechanistic pavement design and evaluation procedures, a repeated load triaxial cell[7,9,12,19,24,42] has been used most frequently. The test apparatus and procedure are explained in detail in References 9, 19, and 42 and are not presented in this report. The hollow cylinder simple shear test[64] is in the development stage at the University of Kentucky in cooperation with the Air Force. Barksdale and Hicks[48] have presented the advantages, disadvantages, approximate cost, and selected sources of four testing systems in a tabular form (Table 4.16). The testing systems described are the mechanical,[23,98] pneumatic,[24,99] open-loop hydraulic system,[12] and closed-loop hydraulic servosystems.[99] The following material is quoted from Barksdale and Hicks:[48]

> For routine testing of pavement materials, a reliable system which is easy to maintain and repair is essential. Furthermore, if the dynamic material properties of all layers of a flexible pavement are to be evaluated, the system should have as minimum requirements a load capacity of at least 1500 to 2000 lbs and the capability of applying a pulse to the specimen in 0.1 sec or less and at frequencies ranging from approximately 0.5 to at least 5 Hz.

The closed loop hydraulic servosystems have by far the best overall capability. These systems, however, can be "electronic monsters" which often are quite expensive and time consuming to keep working properly for routine operations. As a result they are not considered to be suitable as a production type testing system for use in most highway materials laboratories. The pneumatic testing system (or a slightly faster acting air/oil system) does not have nearly the overall capability as that of a closed loop testing system. However, if properly designed, it can meet the minimum requirements for the dynamic testing of pavement materials and is very reliable and easy to maintain. Because of its relatively low cost and high degree of reliability, the pneumatic (or air/oil) type system is recommended for routine production type dynamic testing. Where loads in excess of 4000 to 5000 lb are required such as for loading prototype pavement systems, an open loop hydraulic system can often be used to good advantage.

4.5.2 TEST METHODS

In the conventional triaxial test, a cylindrical specimen is placed in a cell and subjected to repeated deviator stress pulses. Most tests have been conducted using a constant cell pressure σ_3 which is easy to perform. A few researchers[19,100-102] have subjected the specimen to simultaneous repeated axial and lateral stress states which duplicate more closely the stress conditions existing in the field.

4.5.2.1 Resilient Modulus. The resilient modulus is defined as the repeated axial stress in triaxial compression divided by the recoverable axial strain. A detailed discussion by Barksdale and Hicks[48] on the techniques and difficulty of measuring the resilient modulus in triaxial tests is presented in the following two paragraphs.

In any dynamic test, the deformation as well as load must be measured using electronic measuring and recording equipment. Many times, undesirable deformations can occur during loading in the piston and end platens and also in the associated connections between components. If these movements are included in the deformation used to calculate the recoverable strain, the calculated resilient modulus will be smaller than the actual value. Results indicate when the resilient

modulus is greater than about 15,000 psi special measuring devices should be used inside the cell to eliminate this problem. Reasonably reliable resilient deformation measurements can be obtained by attaching two thin, circular aluminum or plexiglass clamps around the specimen at approximately the quarter points. Theoretical studies[100] indicate that the stiffening effect of the clamp should not increase the resilient modulus by more than about 10 to 15 percent.

Reliable axial deformations of the specimens can be obtained using two LVDT's attached to the clamps, placed on opposite sides of the specimen, and wired so that their electrical outputs are added together (or averaged) and then recorded on a reasonably fast-responding electronic recorder. At Georgia Tech, deformations are usually measured using a d-c electrical system by means of a pair of Collins SS-203 or SS-204 LVDT's. Measurements can also be obtained using a-c recording systems. The advantage of using an a-c measuring system is that the LVDT's are lightweight and cost only about one third that of d-c LVDT's. An a-c recording system should not, however, be used without suitable correction networks when phase angle relationships are to be measured between stress and strain. Of course, many other types of measuring systems can be successfully used such as displacement potentiometers and optical scanners.

4.5.2.2 Resilient Poisson's Ratio. Resilient Poisson's ratio is a difficult elastic constant to reliably evaluate for most pavement materials. For an ideal, isotropic, cylindrical specimen of material subjected to a uniform principal stress state, Poisson's ratio is equal to

$$\nu = -\varepsilon_l/\varepsilon_a \qquad (4.22)$$

where ε_l and ε_a are the lateral and axial strains, respectively. Dehlen[100] has theoretically shown that if perfectly frictionless caps and bases are used, the errors associated with uneven lateral strain of the specimen should be less than 10 percent. Therefore, Equation 4.22

can be used to calculate Poisson's ratio provided that end friction is eliminated.

From physical considerations, ν for elastic isotropic materials should be between -1 and 1/2. However, experimentally determined values of ν from the repeated load test with constant confining pressure have been found in some instances to be greater than 1/2.[9] These large values of ν may at least partially be a result of the nonuniform stress and deformation conditions which exist in the triaxial specimens and may also be due to the fact that pavement materials do not behave as ideal elastic solids. However, it can also be assumed that these results indicate anisotropic behavior on the part of the granular materials. The same conclusions were reached by other researchers.[45,100,102] In the variable confining pressure tests conducted by Allen,[19] however, the anisotropic behavior was not observed as evidenced by the consistently lower values of Poisson's ratio.

The following discussion on measuring elastic Poisson's ratio is taken from Reference 48.

> Most researchers[42,9,99,100,103] who have attempted measurements of ν have used either wire resistance strain gages for stabilized materials or LVDT's for nonstabilized materials. For bound materials,[9,42,100] a pair of strain gages can be bonded to the specimen at midheight with the gage oriented horizontally.[9,100] The measurement of lateral deformation in clay and/or unbound gravel has been carried out using two a-c transducers fixed to aluminum or plexiglass clamps at the quarter points.[9] For clays, Dehlen[100] also drilled diametral holes through the sample and used an LVDT to measure the lateral deformation. Another approach to measure lateral deformation is by the use of a lateral deflectometer.[104] This consists of three thin metal probes which press against the specimen and are supported on an aluminum ring positioned about the center of the specimen. A strain gage is bonded to the side of each probe to measure the strain in it as the specimen deforms.
>
> Poisson's ratio can also be determined by recording the total volume change which the specimen undergoes. From the theory of elasticity, Poisson's ratio is related to the volume change by the following approximate relationship:

$$\nu = \frac{1}{2}\left[1 - \frac{1}{\varepsilon_a}\frac{\Delta V}{V}\right] \tag{4.23}$$

where

ν = Poisson's ratio

ε_a = axial strain

ΔV = change in volume of the specimen

V = initial volume of the specimen

The volume change can be evaluated by measuring the deformation profile of the specimen directly or by filling the cell with a fluid and measuring its change in volume.[103]

4.5.2.3 Remarks on Laboratory Tests. The purpose of laboratory testing is to determine the response of pavement materials to the actual traffic loads. Since an element in a pavement system can be subject to any arbitrary combination of normal and shear stresses which may vary with time, the testing apparatus should be designed with the capability of applying arbitrary normal stresses and also permit the application of shear stresses in the presence of these normal stresses. Furthermore, it should be possible to vary these stresses with time. The triaxial compression test most frequently used at the present time does not fulfill these requirements. As was commented by Dehlen,[100]

> Although the triaxial tests had adequately defined the constitutive relations for the materials at points beneath the axis of symmetry, the results were inadequate for conditions elsewhere in the pavement, because the triaxial test permitted only two normal stresses to be varied independently and the resulting two strains measured, while the complete characterization of the materials outside the axis of symmetry required three normal and one shear stress and the resulting four strains.

Table 4.1

Summary of Laboratory Tests to Evaluate the Resilient Properties of Granular Materials (after Seed et al.[34])

Type of Rep. Loading	Reference	Material Investigated	Factors Investigated	Confining Pressure PSI	Deviator Stress PSI	Frequency and Duration	Number of Load Applications	Modulus of Resilient Deformation PSI
Triaxial-compression; repeated-load tests	Seed and Chan[28]	Silty sand	Duration of stress applications	14.7	23.5, 36.0 36.0 36.0	20 per min for ⅓ sec, 2 min on, 2 min off, 20 min on, 20 min off	10,000	21,300, 27,300 23,200 23,000
	Haynes and Yoder[22]	AASHO base course material adjusted gradation to ¾-in. max. size 1. Gravel 2. Crushed stone	Percent of fines passing No. 200 sieve: 6.2, 9.1, 11.5% Degree of saturation: 70, 85, 100%	15.0	55	40 per min	100,000	28,000–63,000
	Univ. of California[26]	Niles aggregate, fairly rounded ¾-in. max. size, 5% passing No. 200 sieve	Void ratio Confining pressure	14.2, 28.4	20, 40, 60	20 per min	10,000	16,700–54,500
	Biarez[10]	Uniform sand, 0.016-in. diam. particles	Variation of E with applied mean normal stress	Mean normal stress, 2.2–145 psi		Cyclic load (rate of deformation not indicated)	~5	At $\sigma_m = 2.2$ psi, 4300 psi At $\sigma_m = 145$ psi, 71,000 psi
	Trollope, Lee, and Morris[11]	Poorly graded dry sand	Initial dry density (loose and dense); rate of deformation from 0.003 to 0.2 in. per min; lateral pressure at constant stress; effect of stress level at constant confining pressure	15–45	Stress level varied	Cyclic load, rate of deformation from 0.003 to 0.02 in. per min	~100	35,000–95,000
	Texas Transp. Institute, Dunlap[12]	Graded material, 1-in. max. size; 6% passing No. 200 sieve; molding water content, 5.5%	Variation of modulus with confining pressure	3–30	3.45 and 51.8	30 per min, duration 0.2 sec	130,000	30,000–160,000 psi
Resiliometer	Hveem, et al.[30]	Range of base course materials used in pavement construction	Relationship between laboratory measurement of resilience of soils and transient pavement deflections	3	10 to 50	8 per min for 0.75 sec	Dependent on nature and condition of material being tested	Resiliency measured in in.³ on 4-in. high by 4-in. diameter specimens
Materials compacted and tested in rigid molds	Coffman, et al.[23]	Aggregate base and subbase, AASHO Test Road	Density, water content, confining pressure, frequency of loading	Base, 14; subbase, 9	Base, 42; subbase, 32	Creep tests; use of transforms to obtain complex modulus in frequency range 0.16–16 cycles per sec	1	Base, 7,500–20,000 Subbase, 5,000–28,000
	Univ. of California	Granular base course mat. ¾-in. max. size, 10% passing No. 200 sieve.	Total and resilient deformation	Compacted and tested in a 6-in. diam. mold	20, 30 and 45; seating load, 3.5 psi	5 per min, duration 0.33 sec	12,000	22,000–48,000
	Barkan[31]	Various types of soil	Water content, grain size, porosity of sand	Tested in a consolidometer	0–70	Not given	Not given	7,700–12,000
	White[32]	Crushed stone, 2½-in. max. size; river gravel, 2½-in. max. size	Gradation, water content, density, applied load	Compacted and tested in steel molds	100 and 200 psi	Load maintained until movement ceased	50, 3,000	40,000–125,000

Table 4.2

Mean Resilient Deformations of AASHTO Road Test Pavements; 1959 and 1960 (after Reference 35)

Loop	Structural Thickness, in.	Resilient Deformation, in.			
		Total		Subgrade	
		Spring	Summer	Spring	Summer
3	15	0.027	0.024	0.018	0.018
4	19	0.031	0.027	0.020	0.020
5	24	0.029	0.025	0.017	0.016
6	27	0.040	0.030	0.024	0.018

Table 4.3

Rutting in the Various Layers of AASHTO Road Test Pavements; 1960 (after Reference 35)

Loop	Time of Year	Rut Depth, in.			
		Surface	Surface & Base	Surface, Base, & Subbase	Subgrade
3	Spring	0.20	0.24	0.64	0.13
	Summer	0.41	0.59	1.21	0.20
4	Spring	0.33	0.39	1.20	0.03
	Summer	0.50	0.62	1.03	0.06
5	Spring	0.49	0.50	1.21	0.20
	Summer	0.46	0.72	1.31	0.13
6	Spring	0.05	0.38	0.83	0.20
	Summer	0.41	0.75	1.38	0.20

Table 4.4

Summary of Material Characteristics of Bases Tested in the Repeated Load Triaxial Apparatus

Base	Description	Grain Size Distribution percent passing					Maximum Density			Soil Classification		
		1/2 in.	3/4 in.	No. 10	No. 60	No. 200	Method*	γ_d pcf	ω_{opt} percent	LL	PI	Classification**
1	Orange-tan, slightly clayey, silty sand	100	100	100	63	40	GHD-7	115.4	13.0	22	6	A4(1); SM-ML
2	40% silty fine sand and 60% No. 467 Crushed Granite Gneiss	99	85	42	25	13	GHD-49	138	4.2	SIC†	N.P.	A-2-4(0); SM
3	40% silty sand and 60% No. 467 Crushed Biotite Gneiss	100	72	39	23	11	T-180C	138	7.5	SIC	N.P.	A-2-4(0); SM
4	17% silty sand and 83% Crushed Biotite Granite Gneiss	95	60	30		13	GHD-49 T-180C	143 141.6	4.6 5.9			A-2-4(0); SM
5	21% sandy silt and 79% Crushed Biotite Granite Gneiss	97	78	28	28	14.8	GHD-49 T-180C	140 140	6.0 6.3	35	15	A-6(5); ML
6	Crushed Prophyritic Granite Gneiss - 3% fines Source A	100	60	25	9	3	GHD-49 T-180C	136 137.4	3.7 6.5			
7	Crushed Porphyritic Granite Gneiss - 11.25% fines. Source A	100	90	45	27	11.25	GHD-49 T-180C	135 135	5.7 6.0			
8	Crushed Biotite Granite Gneiss - 3% fines Source B	100	60	25	9	3	T-180C	137.4	6.5			
9	Crushed Biotite Granite Gneiss - 11.25% fines Source B	100	90	45	27	11.25	T-180C	135	6.0			
10	Crushed Biotite Granite Gneiss - 22% fines Source B	100	90	45	27	22	T-180C	132.9	6.1			

* Maximum density obtained by the State Highway Department of Georgia's test methods GHD-7 and GHD-49 correspond approximately to AASHO designations T-99 and T-180, respectively.

** The AASHO Classification System is given first and the Unified Soil Classification System second.

† Soil slid in cup of Atterberg limit device.

Table 4.5

Summary of Elastic and Plastic Base Characteristics Evaluated from Repeated Load Triaxial Tests

Base	Base Description	Sample Condition	Plastic Strain, 10^2 percent (σ_3 = 10 psi) Deviator Stress Ratio @ 100,000 Repetitions 2.5	3.5	6.0*	Rutting Characteristics: Rut Index for 100,000 Repetitions	Rutting Characteristics: Rut Potential for 1,000,000 Repetitions
1	Silty Fine Sand	100% GHD-7 Soaked	114	00	00	Very Large	---
2	Soil-Aggregate 40-60 Blend	100% GHD-49 Soaked	128 128	270 270	780 ---	1050 (extrapolated)	1130
3	Soil-Aggregate 40-60 Blend	100% T-180C Soaked	58 90	120 190	285 ---	405	467
4	Soil-Aggregate 17-83 Blend	100% GHD-49 Soaked	30 36	82 ---	250 ---	332	371
5	Soil-Aggregate 21-79 Blend	100% GHD-49 Soaked	30 40	44 64	120 ---	164	202
6	Crushed Porphyrite Granite Gneiss - Source A 3% fines	100% T-180C 95% T-180C Soaked	38 50 70	56 76 114	120 170 ---	176	254
7	Crushed Porphyritic Granite Gneiss - Source A 11.25% fines	100% T-180C 95% T-180C Soaked	38 126 170	72 large large	226 --- ---	298	360
8	Crushed Biotite Granite Gneiss - Source B 3% fines	100% T-180C	36	78	100	258	292
9	Crushed Biotite Granite Gneiss - Source B 11.25% fines	100% T-180C	46	105	280	385	630
10	Crushed Biotite Granite Gneiss - Source B 22% fines	101.4% T-180C	46	105	314	419	5520

* These values were in most instances extraploated from laboratory test data.

Table 4.6

Percent Total Axial Strain Accumulated During VCP and CCP Tests

Specimen	Total Plastic Axial Strain ε_p in./in.	Percent ε_p During VCP Test	Percent ε_p During CCP Test
HD-1	0.0036	49	51
MD-1	0.0149	22	78
LD-1	0.0191	48	52
HD-2	0.0158	42	58
MD-2	0.0173	43	57
LD-2	0.0204	43	57
HD-3	0.0063	49	51
MD-3	0.0152	46	54
LD-3	0.0193	43	57

Table 4.7

Material Constant Values Proposed for Various Granular Materials by Other Researchers

Material Description	Constants		Reference
Expression: $M_R = K_1 \sigma_3^{K_2}$			
	K_1	K_2	
Dry, partially crushed gravel	10,094	0.580	9
Dry, crushed gravel	13,126	0.550	9
Partially saturated, partially crushed gravel	7,650	0.591	9
Partially saturated, crushed gravel	8,813	0.569	9
Saturated, partially crushed gravel	9,894	0.528	9
San Diego base	12,225	0.54	9
Gonzales Bypass base	15,000	0.48	16
Gonzales Bypass subbase	10,000	0.40	16
Morro Bay base	11,800	0.39	15
Morro Bay subbase	6,310	0.43	15
Expression: $M_R = K_1' \theta^{K_2'}$			
	K_1'	K_2'	
San Diego base	3,933	0.61	9
Dry, crushed gravel	2,156	0.71	9
Partially saturated, crushed gravel	2,033	0.67	9
Morro Bay subbase	2,900	0.47	15
Morro Bay base	3,030	0.50	15
Expression: $M_R = K_3 + 2K_4 \sigma_r$			
	K_3	K_4	
Crushed limestone	4,856	390	12
Crushed limestone after 36,000 repetitions	37,710	1082	12

Table 4.8

Selected Measured Dynamic Moduli for Unbound Granular Materials (From Dorman and Metcalf)

Test Method	Material Description	Frequency/ Duration, cpm	Load Repetitions	Dynamic Modulus, psi
Repeated load triaxial[16]	State of Colorado, standard base 1/2 in. maximum, 8.7% < #200 Standard subbase, 2-1/2 in. maximum, 79% < #200	120 cpm 0.2 sec	10,000	$E_R = 10{,}618\ \sigma_3\ 0.447$ @ 2.4% w/c $E_R = 10{,}019\ \sigma_3\ 0.465$ @ 6.3% w/c $E_R = 8.687\ \sigma_3\ 0.496$ @ 8.2% w/c
Repeated load triaxial[9]	State of California, well graded, subrounded gravel, 3/4-in. maximum, Class 2 aggregate base	30 cpm 0.1 sec	100	Dry [*] [3%], E_R = 10 to 13,000 σ 0.53 [8%], 8-9,000 σ_3 0.59 Partially Saturated: [3%], E_R = 7-10,000 σ_3 0.55 [8%], E_R = 5-7,000 σ_3 0.60
Repeated load triaxial[9]	State of California, well graded, angular crushed stone, 3/4 in. maximum, Class 2 aggregate base	30 cpm 0.1 sec	100	Dry: 3%, E_R = 11-12,000 σ_3 0.57 10%, E_R = 14-15,000 σ_3 0.50 Partially Saturated: 3%, E_R = 9-10,000 σ_3 0.57 10%, E_R = 7.5-9,500 σ_3 0.57
Repeated load triaxial[42]	Soil-Aggregate 17% Silty Sand, 83% Crushed Granite 100% T-180, w/c = 5.1%	33 cpm 0.1 sec	10,000	$E_R = 3836\ \theta^{0.53}$ $E_R = 3145\ \theta^{0.55}$
Repeated load triaxial[24]	State of California, well graded, subrounded gravel 3/4 in. maximum, Class 2 aggregate base (dry)	30 cpm 0.2 sec	10,000	$E_R = 7000\ \sigma_3\ 0.55$
Repeated load triaxial[42]	Silty fine sand 100% AASHTO T-00 40% < #200 w/c = 13.4%	33 cpm 0.1 sec	10,000	$E_R = 1856\ \theta^{0.61}$ $E_R = 3126\ \theta^{0.37}$

* Refers to percent passing the No. 200 sieve.

Table 4.9

Test Specimens for Triaxial Tests

Specimen	Material	Density, pcf	Percent Moisture	Percent Saturation
HD-1	Crushed Stone	138.0 High	5.7	78
MD-1	Crushed Stone	134.0 Intermediate	6.3	73
LD-1	Crushed Stone	130.0 Low	7.0	70
HD-2	Gravel	139.4 High	6.3	82
MD-2	Gravel	134.0 Intermediate	6.5	74
LD-2	Gravel	131.0 Low	6.7	69
HD-3	Blend	139.5 High	6.3	88
MD-3	Blend	134.5 Intermediate	6.8	78
LD-3	Blend	131.0 Low	7.2	74

Table 4.10

Regression Equations for E_r from Primary Test Data

Specimen	Material Type	Test Type	$E_r = f(\theta)$	Correlation Coefficient	Standard Error	$E_r = f(\sigma_3)$	Correlation Coefficient	Standard Error
HD-1	Crushed Stone	VCP	$6635\theta^{.40}$	.930**	3144	$18010\sigma_3^{.28}$	.669*	6338
MD-1	Crushed Stone	VCP	$1793\theta^{.70}$	.992**	1463	$8556\sigma_3^{.57}$	.794**	7014
LD-1	Crushed Stone	VCP	$2113\theta^{.65}$	.982**	2058	$8410\sigma_3^{.57}$	.819**	6227
HD-2	Gravel	VCP	$7766\theta^{.32}$	.767**	3996	$18480\sigma_3^{.19}$	.515(a)	5338
MD-2	Gravel	VCP	$6995\theta^{.33}$	.906**	2202	$15738\sigma_3^{.23}$	.664*	3897
LD-2	Gravel	VCP	$1613\theta^{.69}$	.973**	2033	$7924\sigma_3^{.51}$	.781**	5473
HD-3	Blend	VCP	$6891\theta^{.45}$	.980**	2035	$18951\sigma_3^{.35}$	.832**	5638
MD-3	Blend	VCP	$7725\theta^{.33}$	.981**	1042	$15806\sigma_3^{.26}$	.841**	2890
LD-3	Blend	VCP	$4562\theta^{.43}$	.856**	3367	$14516\sigma_3^{.24}$	.498(a)	5648
HD-1	Crushed Stone	CCP	$2376\theta^{.69}$	.997**	1149	$12454\sigma_3^{.55}$	.845**	7896
MD-1	Crushed Stone	CCP	$4928\theta^{.46}$	.973**	1950	$14254\sigma_3^{.39}$	.872**	4115
LD-1	Crushed Stone	CCP	$3083\theta^{.59}$	.962**	3132	$11068\sigma_3^{.53}$	.909**	4813
HD-2	Gravel	CCP	$4596\theta^{.50}$	.741**	8063	$11128\sigma_3^{.54}$	.803**	7157
MD-2	Gravel	CCP	$8016\theta^{.31}$	.803**	3551	$14729\sigma_3^{.31}$	.838**	3247
LD-2	Gravel	CCP	$2849\theta^{.56}$	.882**	4289	$8517\sigma_3^{.55}$	.916**	3641
HD-3	Blend	CCP	$5989\theta^{.48}$	.932**	4254	$16433\sigma_3^{.45}$	.922**	4542
MD-3	Blend	CCP	$6459\theta^{.37}$	.829**	3977	$13379\sigma_3^{.37}$	.873**	3471
LD-3	Blend	CCP	$2966\theta^{.60}$	.882**	4962	$9079\sigma_3^{.58}$	.914**	4260

(a) Significant at $\alpha = .05$
* Significant at $\alpha = .01$
** Significant at $\alpha = .001$

Table 4.11

Regression Equations for ν_r from Primary Test Data

Specimen	Material Type	Test Type	$\nu_r = f(\sigma_1/\sigma_3)$	Correlation Coefficient	Standard Error
HD-1	Crushed Stone	VCP	$.62 - .19\ (\sigma_1/\sigma_3) + .04\ (\sigma_1/\sigma_3)^2 - .002\ (\sigma_1/\sigma_3)^3$	.907***	.026
MD-1	Crushed Stone	VCP	$.47 - .08\ (\sigma_1/\sigma_3) + .02\ (\sigma_1/\sigma_3)^2 - .001\ (\sigma_1/\sigma_3)^3$	.838***	.045
LD-1	Crushed Stone	VCP	$.60 - .14\ (\sigma_1/\sigma_3) + .02\ (\sigma_1/\sigma_3)^2 - .0007\ (\sigma_1/\sigma_3)^3$	.881***	.036
HD-2	Gravel	VCP	$-.12 + .45\ (\sigma\ /\sigma_3) - .09\ (\sigma_1/\sigma_3)^2 + .005\ (\sigma_1/\sigma_3)^3$	.645***	.085
MD-2	Gravel	VCP	$.46 + .01\ (\sigma_1/\sigma_3) - .01\ (\sigma_1/\sigma_3)^2 + .002\ (\sigma_1/\sigma_3)^3$	.889***	.026
LD-2	Gravel	VCP	$.70 - .22\ (\sigma_1/\sigma_3) + .04\ (\sigma_1/\sigma_3)^2 - .002\ (\sigma_1/\sigma_3)^3$	.925***	.027
HD-3	Blend	VCP	$.49 + .01\ (\sigma_1/\sigma_3) - .01\ (\sigma_1/\sigma_3)^2 + .001\ (\sigma_1/\sigma_3)^3$	.766***	.037
MD-3	Blend	VCP	$.50 - .02\ (\sigma_1/\sigma_3) - .003\ (\sigma_1/\sigma_3)^2 + .0006\ (\sigma_1/\sigma_3)^3$	.561*	.048
LD-3	Blend	VCP	$.52 - .07\ (\sigma_1/\sigma_3) + .006\ (\sigma_1/\sigma_3)^2 + .0002\ (\sigma_1/\sigma_3)^3$	.840***	.026
HD-1	Crushed Stone	CCP	$-.17 + .30\ (\sigma_1/\sigma_3) - .04\ (\sigma_1/\sigma_3)^2 + .002\ (\sigma_1/\sigma_3)^3$	.895***	.047
MD-1	Crushed Stone	CCP	$.29 + .12\ (\sigma_1/\sigma_3) - .01\ (\sigma_1/\sigma_3)^2 + .0006\ (\sigma_1/\sigma_3)^3$	.746***	.060
LD-1	Crushed Stone	CCP	$-.01 + .28\ (\sigma_1/\sigma_3) - .04\ (\sigma_1/\sigma_3)^2 + .002\ (\sigma_1/\sigma_3)^3$	.723***	.095
HD-2	Gravel	CCP	$-.14 + .46\ (\sigma_1/\sigma_3) - .06\ (\sigma_1/\sigma_3)^2 + .003\ (\sigma_1/\sigma_3)^3$	.429(a)	.208
MD-2	Gravel	CCP	$.95 - .22\ (\sigma_1/\sigma_3) + .04\ (\sigma_1/\sigma_3)^2 - .002\ (\sigma_1/\sigma_3)^3$	.654**	.144
LD-2	Gravel	CCP	$-.04 + .32\ (\sigma_1/\sigma_3) - .05\ (\sigma_1/\sigma_3)^2 + .003\ (\sigma_1/\sigma_3)^3$	.953***	.056
HD-3	Blend	CCP	$-.16 + .37\ (\sigma_1/\sigma_3) - .05\ (\sigma_1/\sigma_3)^2 + .003\ (\sigma_1/\sigma_3)^3$	.868***	.073
MD-3	Blend	CCP	$-.02 + .27\ (\sigma_1/\sigma_3) - .03\ (\sigma_1/\sigma_3)^2 + .001\ (\sigma_1/\sigma_3)^3$	.828***	.091
LD-3	Blend	CCP	$-.09 + .36\ (\sigma_1/\sigma_3) - .05\ (\sigma_1/\sigma_3)^2 + .003\ (\sigma_1/\sigma_3)^3$	.729***	.121

(a) Significant at $\alpha = 0.1$
* Significant at $\alpha = .02$
** Significant at $\alpha = .01$
*** Significant at $\alpha = .001$

Table 4.12

Requirements for Atterberg Limits and Gradations

Material	Maximum Design CBR	Maximum Permissible Value: Size in.	Gradation Requirements, Percent Passing: No. 10	No. 200	LL	PI
Subbase	50	3	50	15	25	5
Subbase	40	3	80	15	25	5
Subbase	30	3	100	15	25	5
Select material	20	3*	---	25*	35*	12*

No.	Type	Design CBR
1	Graded crushed aggregate	100
2	Water-bound macadam	100
3	Dry-bound macadam	100
4	Bituminous intermediate and surface courses, central plant, hot mix	100
5	Limerock	80
6	Stabilized aggregate	80

* Suggested limits.

Table 4.13

Pavements Designed by the CBR Equation

Pavement	Single-Wheel Load, kips	Tire Inflation Pressure psi	Thickness, in. Surface	Thickness, in. Base	Thickness, in. Subbase	Subgrade CBR	Coverages at Failure
1	30	105	3	6	6	4	40
2	30	105	3	6	15	4	913
3	30	105	3	6	24	4	20,000
4	30	105	3	6	6	8	549
5	30	105	3	6	6	12	5,750
6	50	175	3	6	6	4	12
7	50	175	3	6	15	4	125
8	60	207	3	6	24	4	575
9	60	207	3	6	32	4	3,990
10	50	175	3	6	6	12	331
11	60	207	3	6	24	12	408,000
12	60	207	3	6	24	8	22,400
13	50	175	3	6	6	16	1,380

Table 4.14

Dynamic Modulus and CBR Values

Dynamic Modulus E 103 psi (1)	CBR Value (2)
Heukelom[72]	
29.02	14
29.16	12
29.16	8
38.41	25
39.83	21
39.83	21
39.83	15
41.25	23
41.96	31
51.49	50
51.92	38
59.03	60
59.74	21
170.70	160
176.39	160
174.96	160
Jones[75]	
.4.84	2.6
6.26	2.3
6.26	3.4
6.76	2.5
6.76	4.7
7.68	4.4
7.97	6.0
8.68	5.3
8.82	6.0
8.82	7.3
9.67	5.2
9.67	6.0
9.67	6.8
10.24	5.7
10.24	9.3
10.67	6.3
11.95	6.2
12.23	7.2
12.23	15.0
13.51	8.2
Jones[75] (Continued)	
15.36	4.3
18.77	11.4
18.77	14.3
20.20	12.0
21.91	18
Nijboer and Dekruijf[76]	
44.10	16
62.59	40
65.43	34
65.43	37
71.12	28
76.81	78
85.35	123
96.73	87
101.00	31
101.00	45
101.00	67
103.84	96
106.69	85
109.53	50
118.07	170
123.76	67
177.81	170
197.72	150
219.06	100
248.93	164
Foss Field[77]	
35.1	5
138.0	38
174.6	38
220.7	76
226.2	90
129.1	101
13.5	5
30.3	9

(Continued)

Table 4.14 (Concluded)

Dynamic Modulus E 103 psi (1)	CBR Value (2)
Foss Field[77] (Continued)	
34.4	9
59.3	12
57.9	12
101.9	31
64.6	31
54.2	43
68.0	43
71.0	52
79.6	52
13.1	5
15.1	5
18.1	5
20.4	5
20.7	6
21.5	6
17.1	7
17.1	7
19.5	7
19.5	7
28.2	10
29.2	10
AASHTO[78]	
124.0	59.0
177.0	19.0
173.0	20.0
313.0	157.0
338.0	124.0
145.0	25.5
57.0	26.0
49.0	10.2
165.0	105.0
136.0	48.0
AASHTO[78] (Continued)	
147.0	73.0
148.0	49.0
31.0	3.2
40.0	3.2
29.0	3.4
31.0	3.3
22.0	3.8
18.0	4.0
48.0	3.2
50.0	3.6
42.0	4.2
Texas Highway[79]	
154.5	129
316.0	134
141.2	185
194.9	149
218.7	165
183.3	202
61.2	20
68.2	26
58.4	85
107.9	61
23.7	5
27.3	6
30.6	17
40.4	13
33.5	10
29.6	14
25.0	19
77.8	10
48.7	11

Table 4.15

List of Pertinent Variables Affecting Material Response (from Deacon[97])

I. Loading variables
    A. Stress history (nature of prior loading)
        1. Nonrepetitive loading (such as preconsolidation)
        2. Repetitive loading
            a. Nature, whether simple or compound
            b. Number of repetitive applications (repetitions)
    B. Initial stress state (magnitude and direction of normal and shear stresses)
    C. Incremental loading
        1. Mode of loading
            a. Controlled stress (or load)
            b. Controlled strain (or deformation)
            c. Intermediate modes
        2. Intensity (magnitude and direction of incremental normal and shear stresses)
        3. Stress path (relation among stresses, both normal and shear, as test progresses)
        4. Time path
            a. Static
                (1) Constant rate of stress (or load)
                (2) Constant rate of strain (or deformation)
                (3) Creep
                (4) Relaxation
            b. Dynamic
                (1) Impact
                (2) Resonance
                (3) Other, including sinusoidal (rate of loading is variable) and pulsating (duration, frequency, and shape of load curve are variables)
        5. Type of behavior observed
            a. Strength (limiting stresses and strains)
            b. Deformability
        6. Homogeneity of stresses
        7. Drainage

II. Mixture variables
    A. Mineral particles
        1. Maximum and minimum size
        2. Gradation
        3. Shape
        4. Surface texture
        5. Angularity
        6. Mineralogy
        7. Adsorbed ions
        8. Quantity

(Continued)

Table 4.15 (Continued)

- B. Binder
  - 1. Type
  - 2. Hardness
  - 3. Quantity
- C. Water (quantity)
- D. Voids
  - 1. Quantity
  - 2. Size
  - 3. Shape
- E. Construction process
  - 1. Density
  - 2. Structure
  - 3. Degree of anisotropy
  - 4. Temperature
- F. Homogeneity

III. Environmental variables
- A. Temperature
- B. Moisture
- C. Alteration of material properties with time
  - 1. Thixotropy
  - 2. Aging
  - 3. Curing
  - 4. Densification

Test configurations are listed in the following outline.

I. Tension
- A. Uniaxial tension
- B. Indirect (splitting) tension
- C. Cohesiometer

II. Compression
- A. Unconfined, uniaxial compression
- B. Triaxial compression
  - 1. Open system
    - a. Isotropic compression
    - b. Conventional triaxial compression, whether normal, vacuum, or high-pressure
    - c. Box with cubical specimen
  - 2. Closed system
    - a. Oedometer
    - b. Cell
    - c. Hveem stabilometer

III. Flexure
- A. Rotation
  - 1. Rotating
  - 2. Nonrotating

(Continued)

Table 4.15 (Concluded)

- B. Loading
  - 1. Cantilever
  - 2. Simple beam
    - a. Point support
    - b. Uniform support

IV. Direct shear
- A. Direct shear (rigid-split box)
- B. Double direct shear
- C. Uniform direct shear (rigid caps with confined rubber membrane and split rings for lateral restraint)
- D. Uniform strain direct-shear (hinged box)
- E. Punching shear

V. Torsion
- A. Pure torsion
- B. Triaxial torsion
- C. Specimen shape
  - 1. Solid cylinder
  - 2. Thick-walled, hollow cylinder

VI. Indirect
- A. Penetration tests
- B. Squeeze tests
- C. Marshall stability
- D. Angle of repose
- E. Others

Possible specimen shapes are enumerated in the following.

I. Rectangular parallelepiped
- A. Short
- B. Long
- C. Cubic

II. Cylinder
- A. Solid
  - 1. Short
  - 2. Long
- B. Thick-walled, hollow
  - 1. Short
  - 2. Long

III. Plate

IV. Other

Table 4.16

Comparison of Dynamic Testing Systems (from Barksdale and Hicks[48])

Testing System	Maximum Response Hz	Advantages and Disadvantages	Cost $1000	Selected Sources
1. Mechanical	5-25	A relatively reliable system. Some problems with the design, balance, and operation of the system. Can apply desired load pulse shapes by adjusting cams.	4-6	Special Fabrication only
2. Pneumatic	5-8	Relatively simple, cheap, and reliable testing system. Easy to design and repair. Will require periodic replacement of valves and cylinder. Practical load limit of about 3,000-5,000 lbs. Hard to apply an exact pulse shape.	3-5	(a) Geotechnical Research, Inc., 2403 Wylie Dr., S. E., Marietta, Ga. 30062 (b) Research Engineering, 2640 Dundee Road, San Pablo. California (c) Structural Behavior Engineering Lab, Inc., P. O. Box 9727, Phoenix, Arizona
3. Hydraulic (open loop control)	5-8	More complex to design and set up than pneumatic system and requires a hydraulic pump and storage reservoir. A hydraulic system has a faster response than a pneumatic one and can go to much higher loads. Hard to apply an exact pulse shape.	4.5-8	Special Fabrication only
4. Hydraulic Servosystem (closed loop control)	25-100	Can have a fast response, high load capacity, and capability to apply any pulse shape to specimens. Disadvantages are its high initial and maintenance cost, and complex electronics. Some systems are very hard to balance and keep in proper operating condition.	10-30	(a) Hydratech, 2890 John R. Rd., Troy, Michigan 48084 (b) MB Electronics, P. O. Box 1825, New Haven, Conn. 06508 (c) MTS Systems Corp., Minneapolis, Minn. 55424

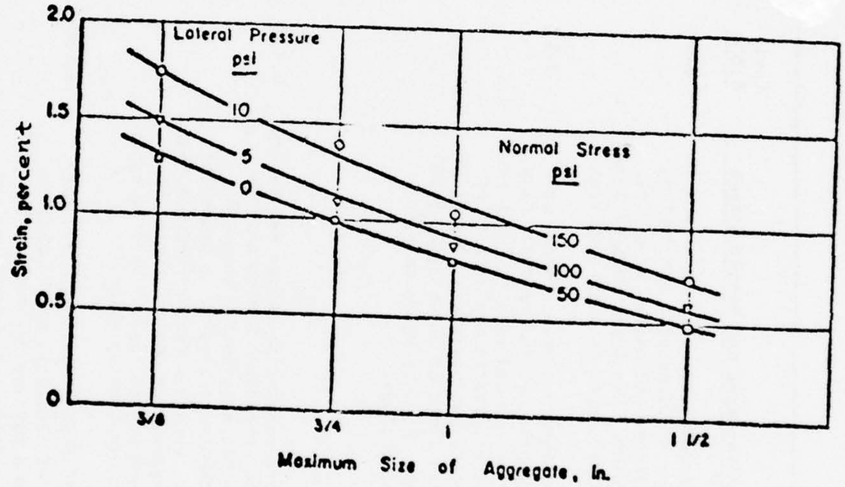

Figure 4.1. Effect of maximum size on rigidity of graded aggregate base (after Kalcheff[2])

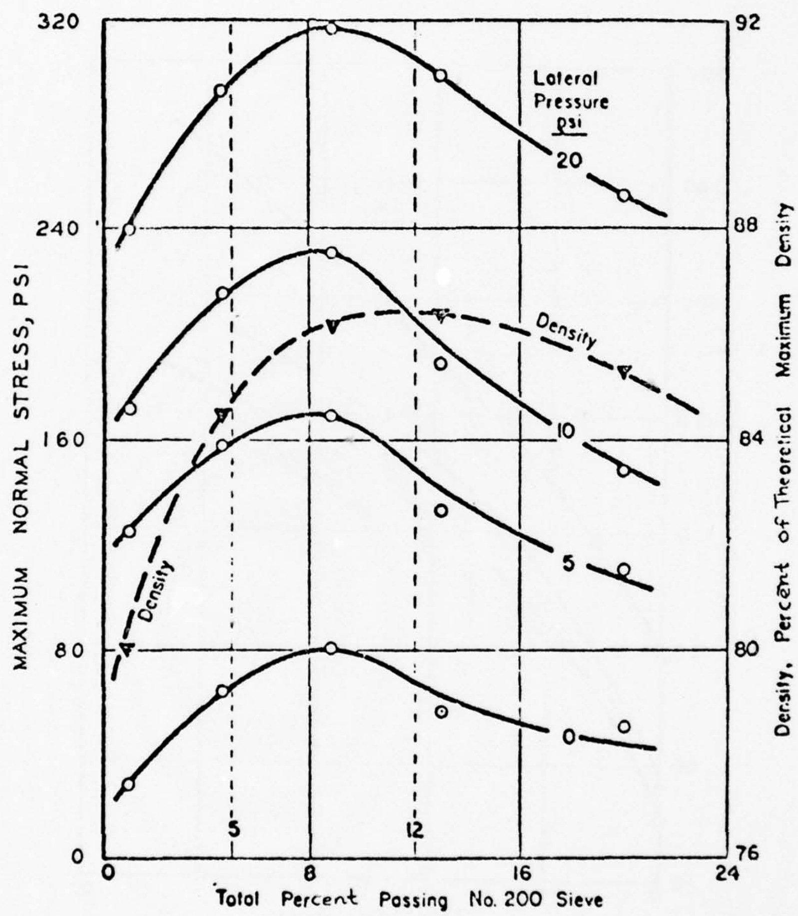

Figure 4.2. Gradation effect on ultimate triaxial strength and density of a continuously graded aggregate base with 3/4-in. nominal size (after Kalcheff[2])

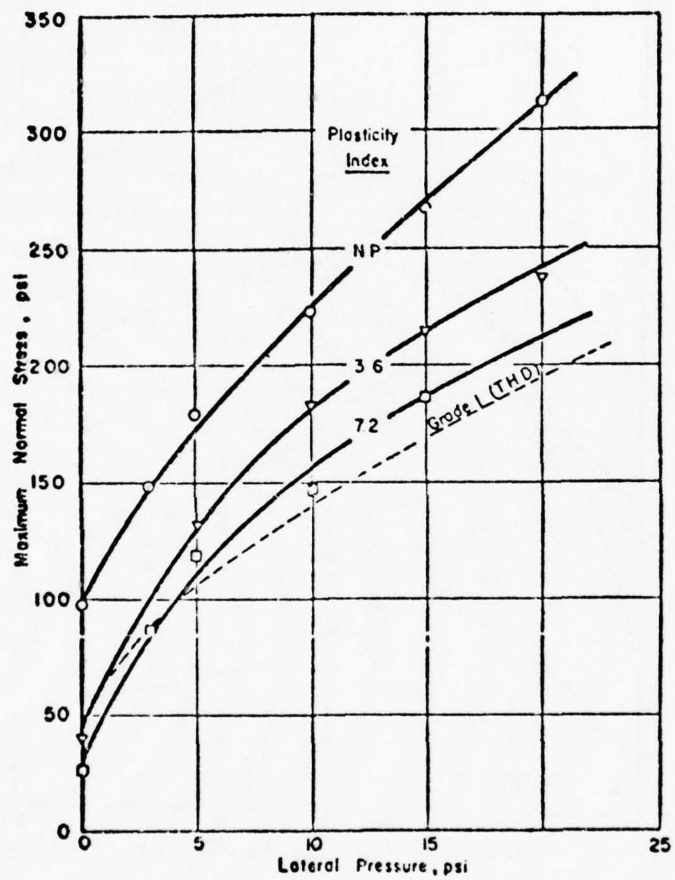

Figure 4.3. Effect of plasticity index on the ultimate triaxial stresses (after Kalcheff[2])

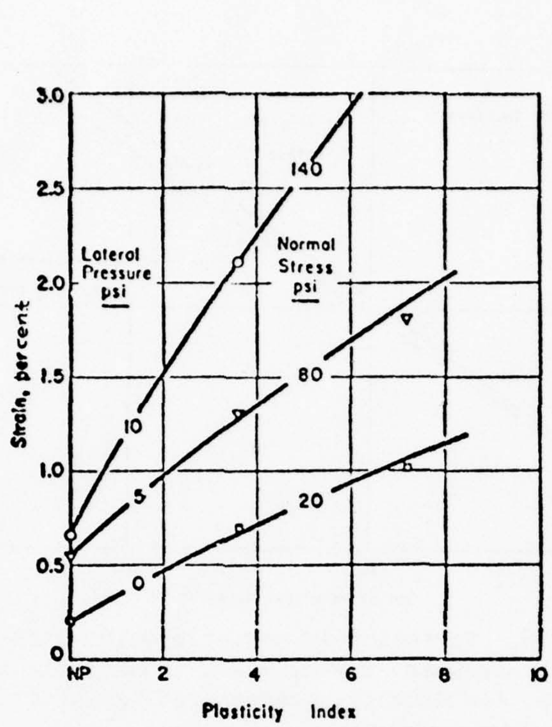

Figure 4.4. Effect of plasticity index on rigidity of graded aggregate base (after Kalcheff[2])

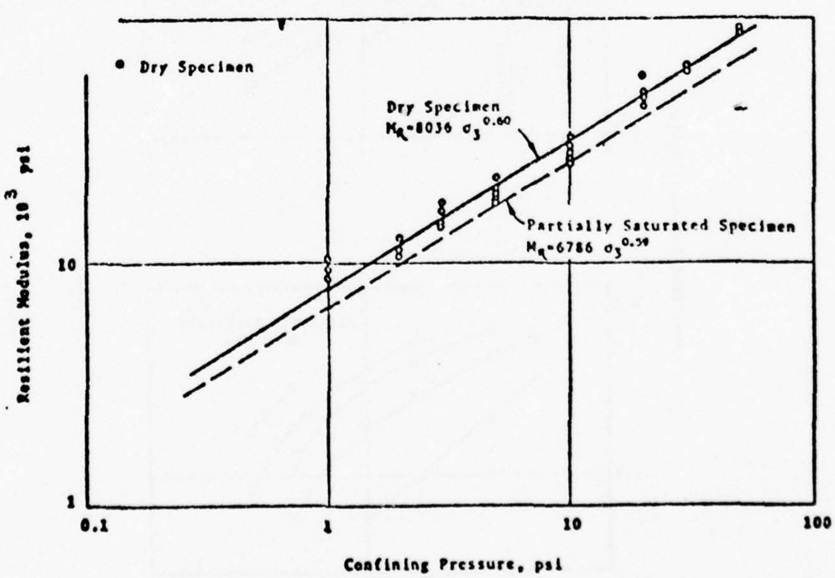

Figure 4.5. Variation in secant modulus with confining pressure; partially crushed aggregate, low density, coarse grading (after Hicks[9])

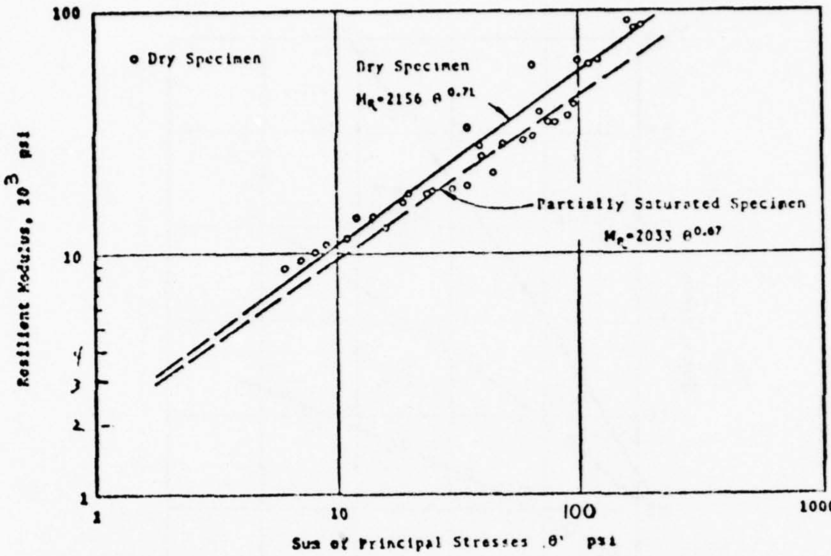

Figure 4.6. Variation in secant modulus with sum of principal stresses $\theta = \sigma_1 + 2\sigma_3$; partially crushed aggregate, low density, coarse grading (after Hicks[9])

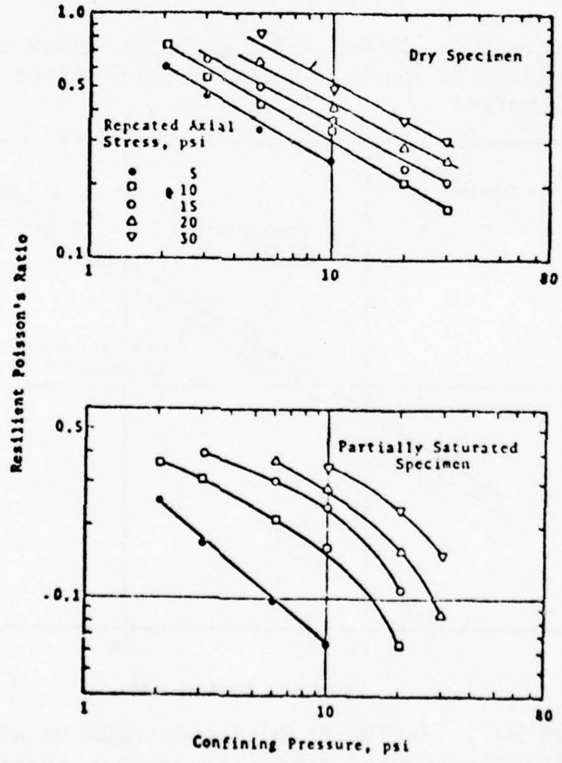

Figure 4.7. Variation in secant Poisson's ratio with stress level; partially crushed aggregate, low density, coarse grading (after Hicks[9])

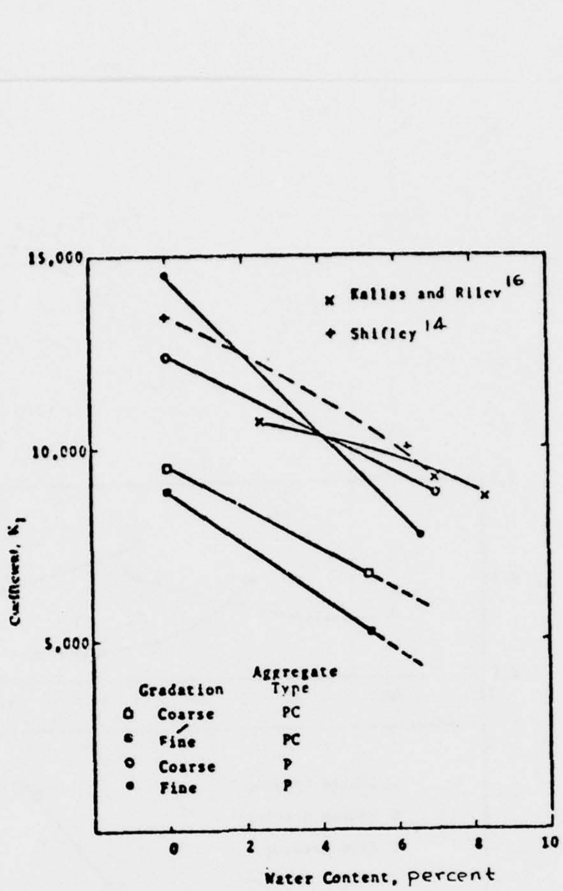

Figure 4.8. Variation in regression constant K_1 with water content in relationship $M_R = K_1\sigma_3^{K_2}$ (after Hicks[9])

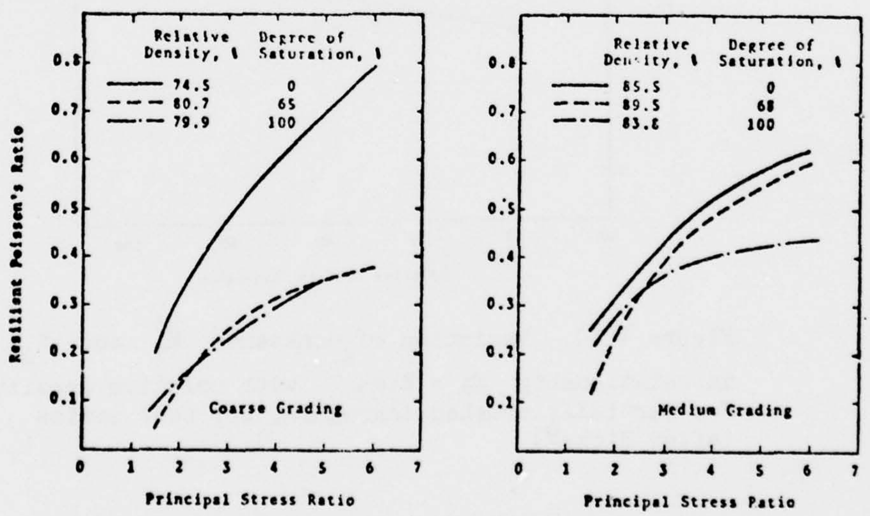

Figure 4.9. Effect of degree of saturation on the relationship between resilient Poisson's ratio and principal stress ratio for partially crushed aggregate (after Hicks[9])

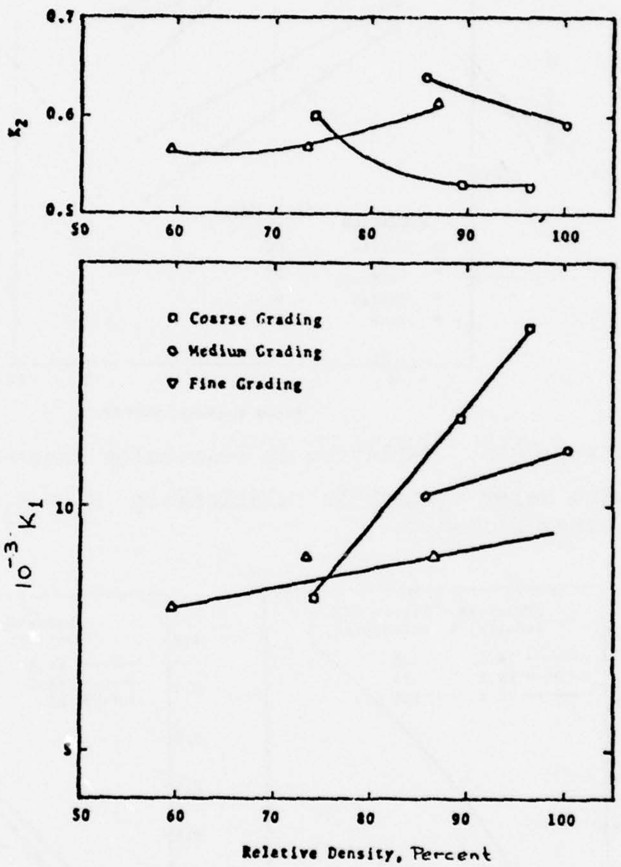

Figure 4.10. Variation of constants K_1 and K_2 in relationship $M_R = K_1 \sigma_3^{K_2}$ with relative density for partially crushed aggregate; dry test series (after Hicks[9])

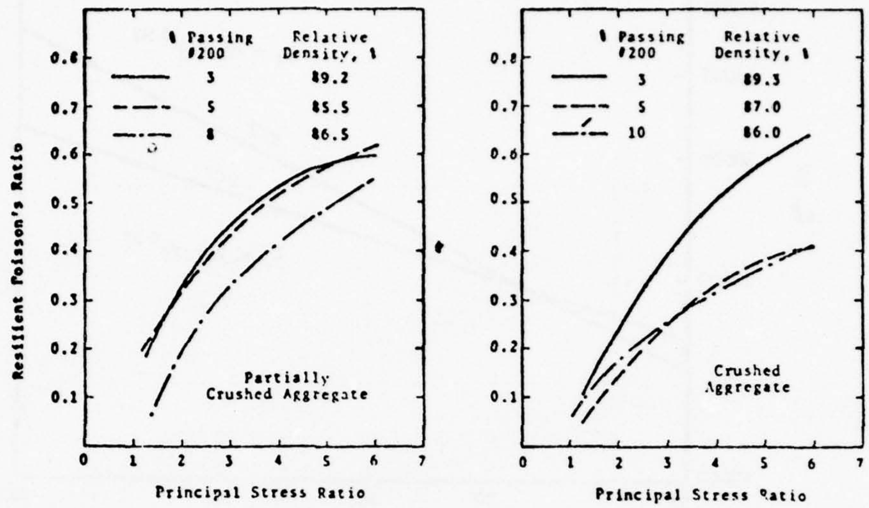

Figure 4.11. Effect of aggregate gradation (percent passing No. 200) on the relationship between resilient modulus and confining pressure σ_3 ; dry test series (after Hicks[9])

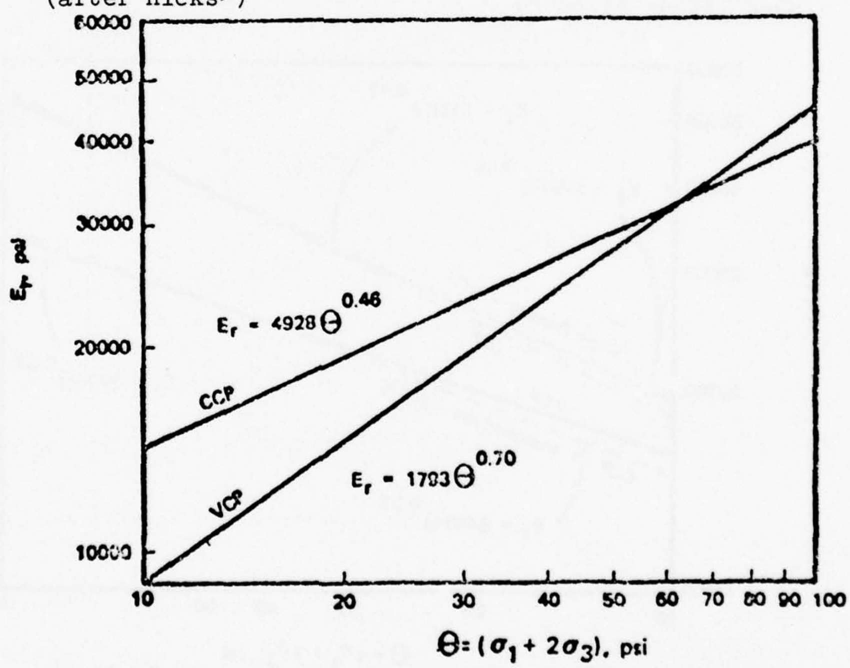

Figure 4.12. Comparison of VCP and CCP relationships between E_r and θ ; intermediate-density crushed stone specimen MD-1 (after Allen[19])

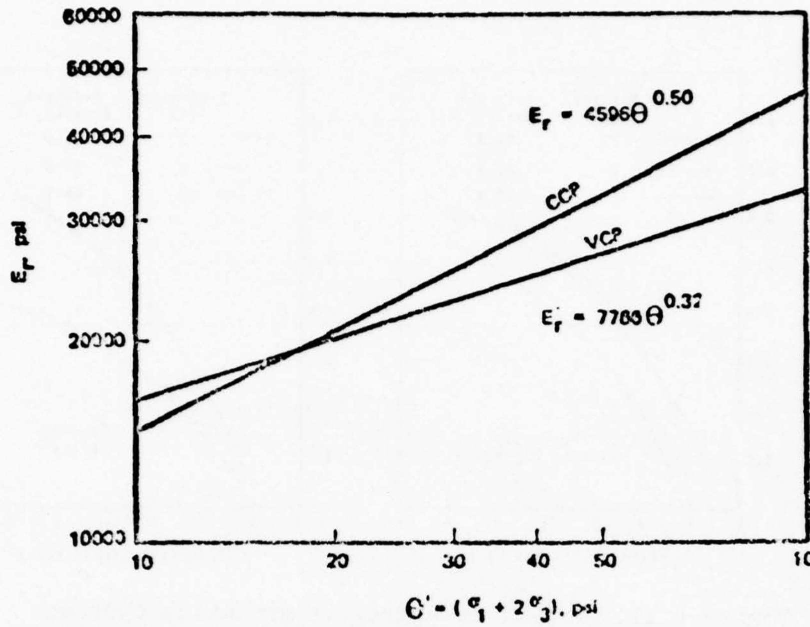

Figure 4.13. Comparison of VCP and CCP relationships between E_r and θ ; high-density gravel specimen HD-2 (after Allen[19])

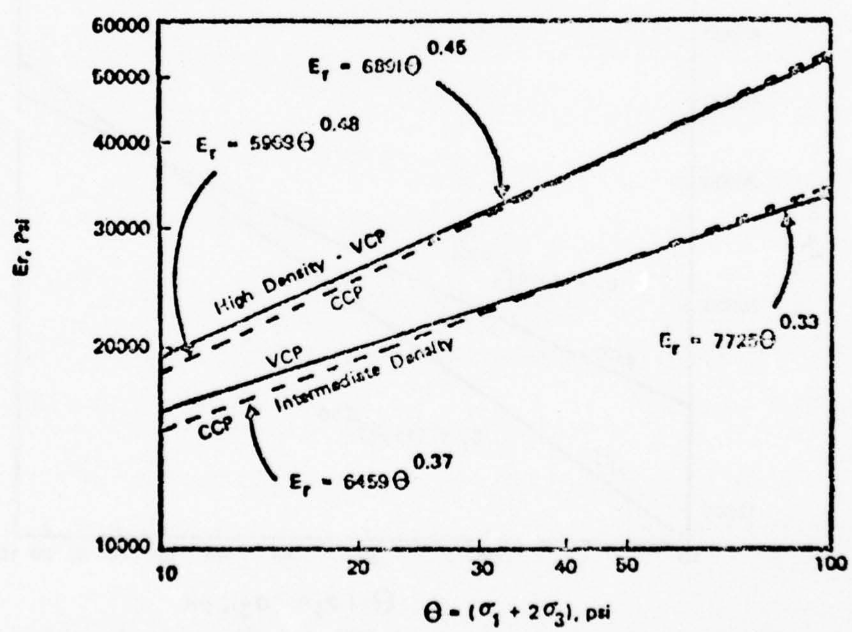

Figure 4.14. Comparison of VCP and CCP relationships between E_r and θ ; high-density (HD-3) and intermediate-density (MD-3) blend specimens (after Allen[19])

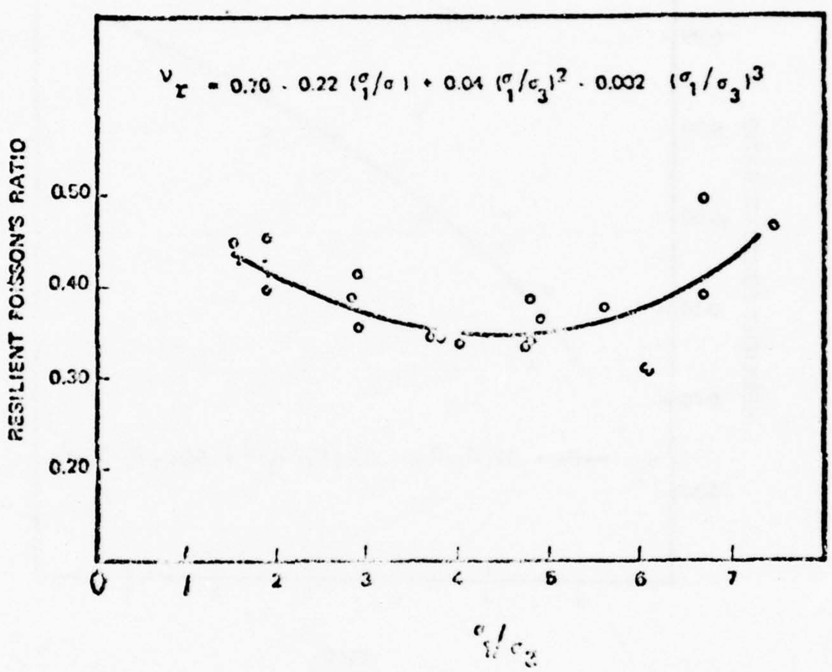

Figure 4.15. Resilient Poisson's ratio ν_r as a function of principal stress ratio (σ_1/σ_3) ; VCP results, low-density gravel specimen LD-2 (after Allen[19])

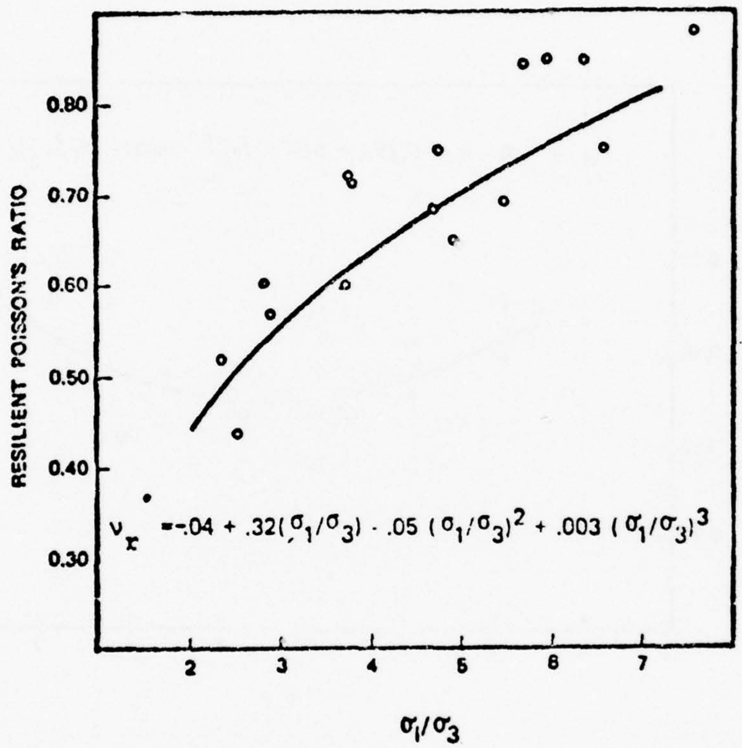

Figure 4.16. Resilient Poisson's ratio ν_r as a function of principal stress ratio (σ_1/σ_3) ; CCP results, low-density gravel specimen LD-2 (after Allen[19])

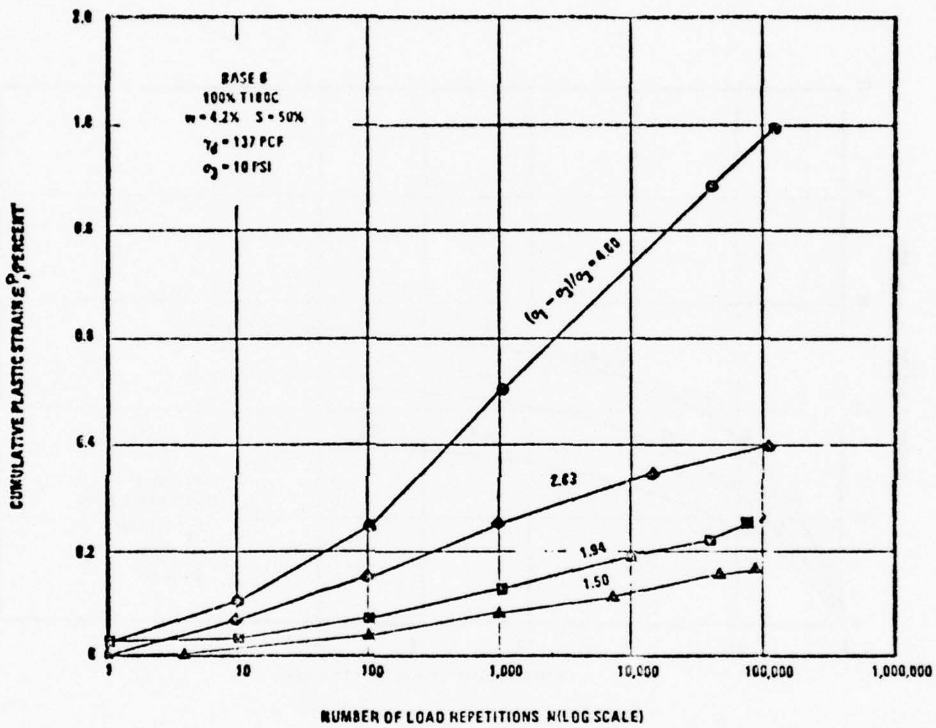

Figure 4.17. Influence of number of load repetitions and deviator stress ratio on plastic strain in a porphyrite granite gneiss; 3 percent fines (after Barksdale[42])

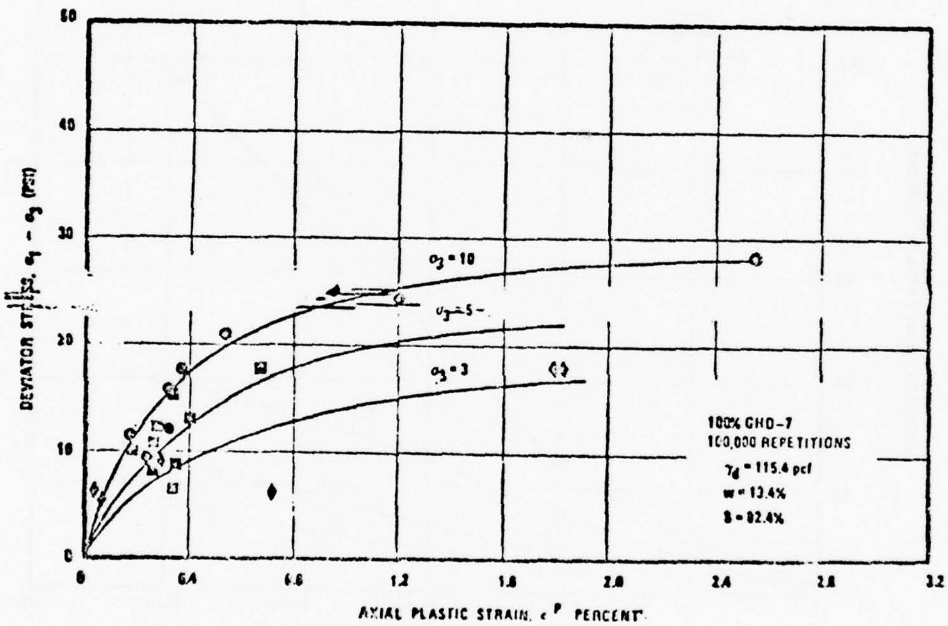

Figure 4.18. Influence of deviator stress and confining pressure on plastic strain after 100,000 repetitions in a fine silty sand; Base 1 (after Barksdale[42])

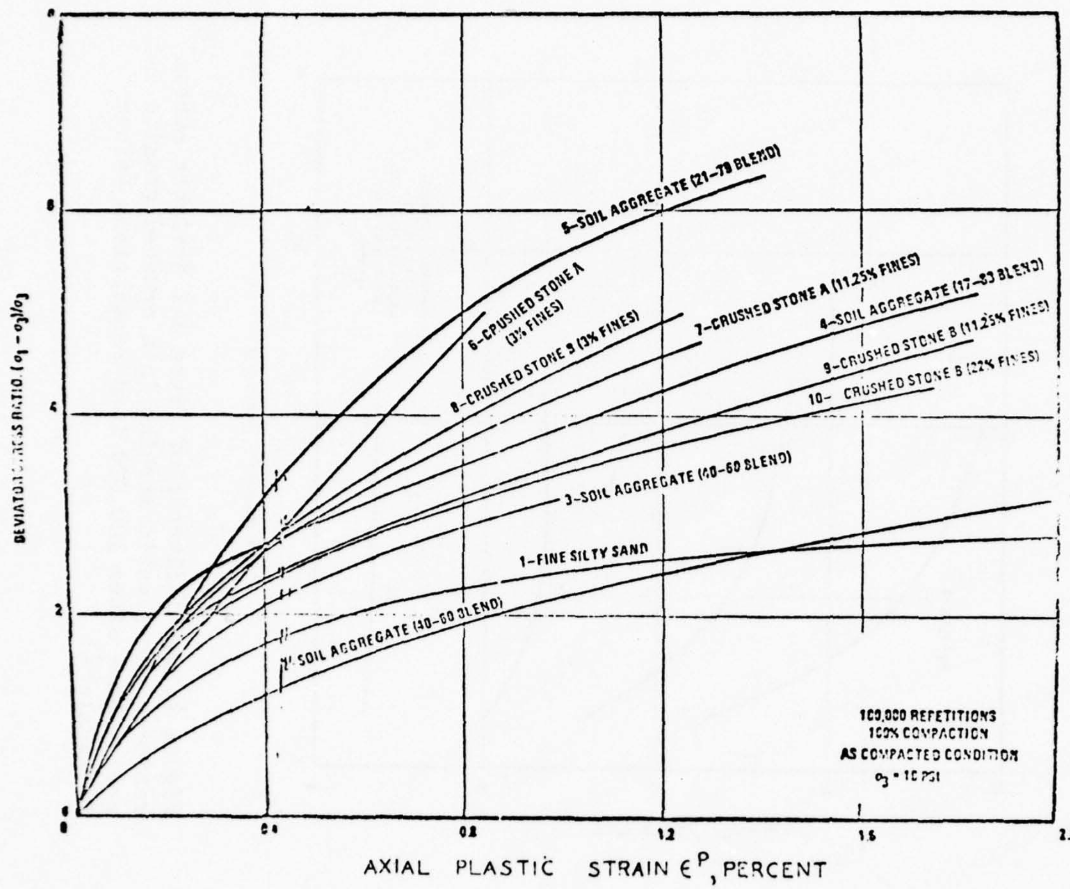

Figure 4.19. Summary of plastic stress-strain characteristics at 100,000 load repetitions and a confining pressure of 10 psi (after Barksdale[42])

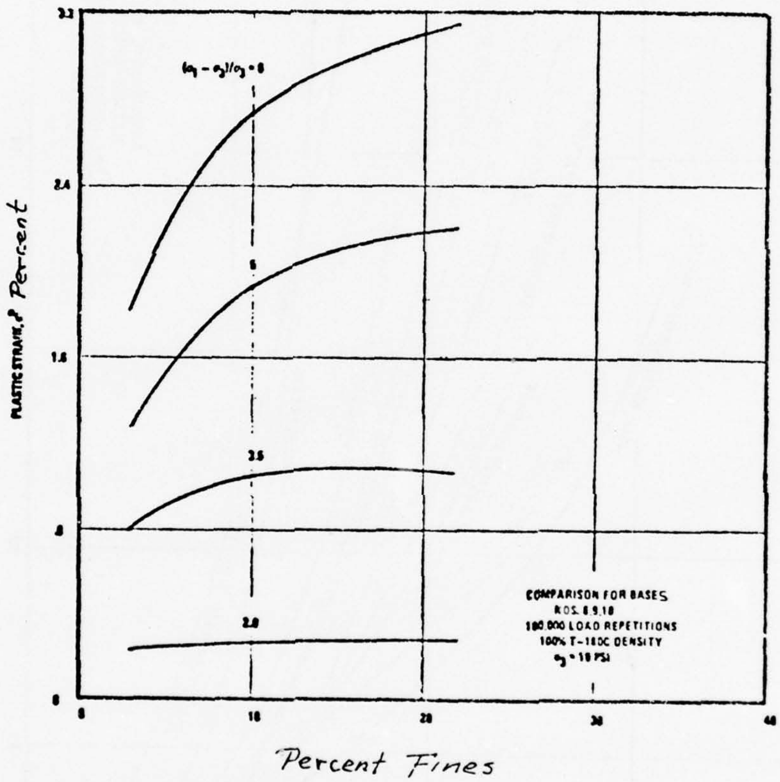

Figure 4.20. Influence of fines and deviator stress ratio on the plastic strains in a crushed granite gneiss base after 100,000 load repetitions (after Barksdale[42])

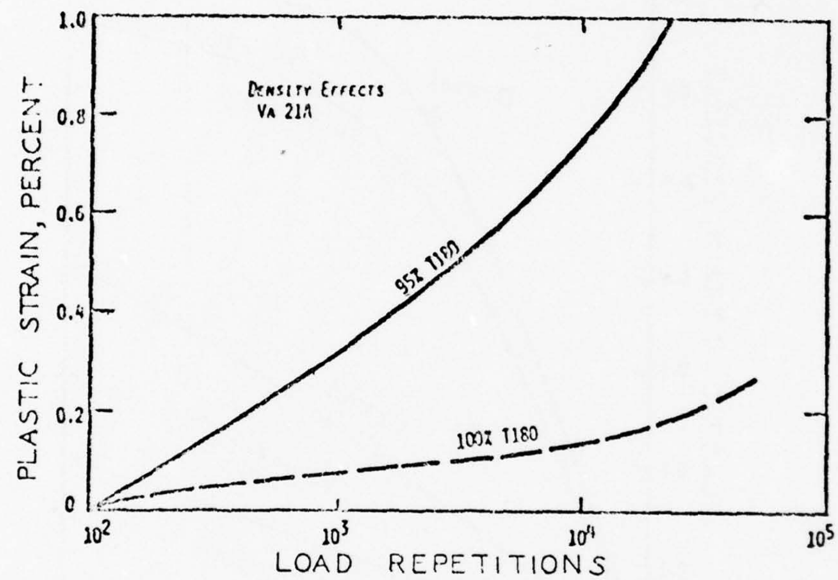

Figure 4.21. Effect of density on the plastic strain accumulation with load repetitions (after Kalcheff[2])

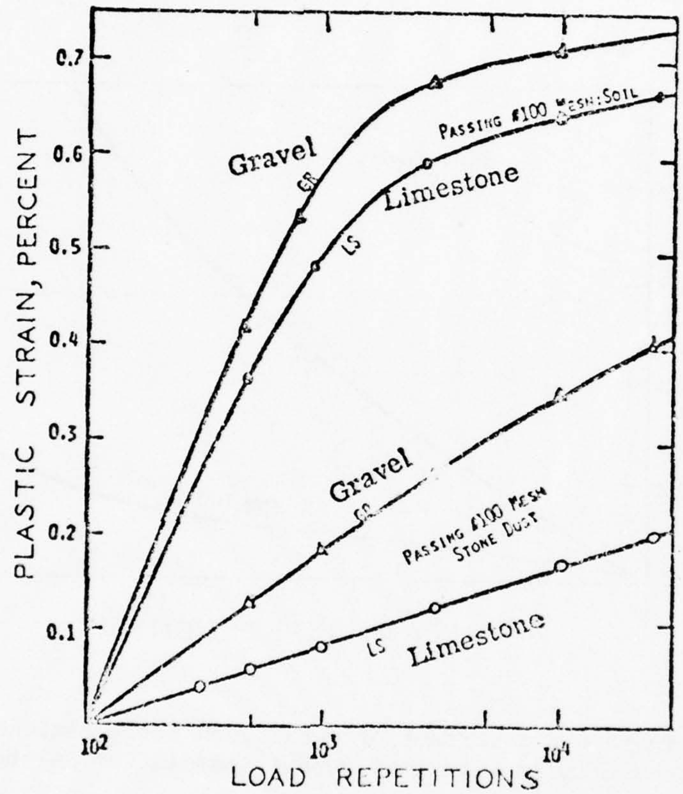

Figure 4.22. Effect of type of fines on the plastic strains of two graded aggregate bases (after Kalcheff[2])

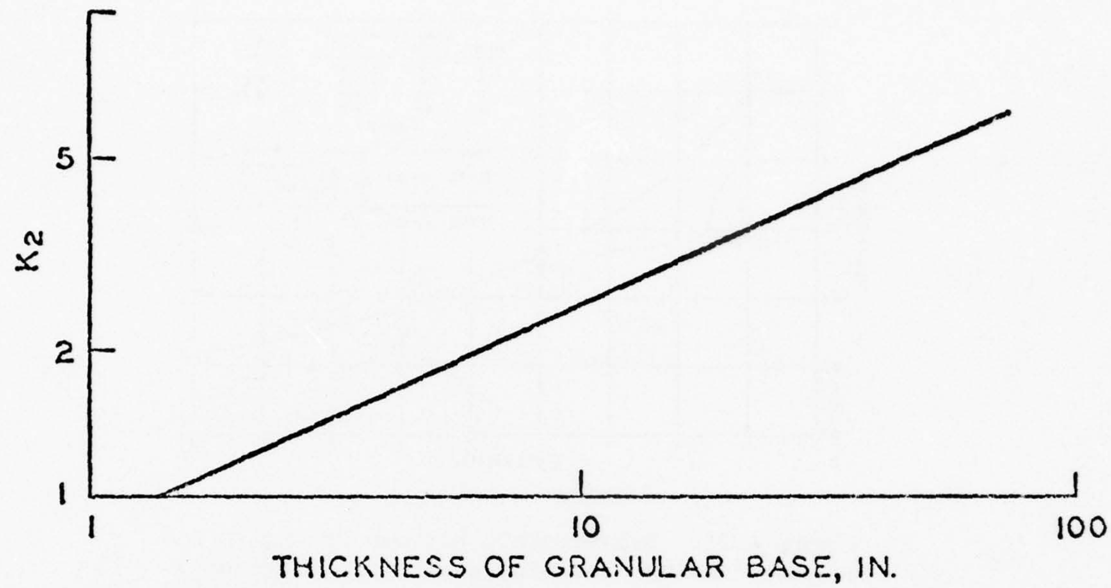

Figure 4.23. Relation of modulus ratio K_2 to thickness of granular base (after Dorman and Metcalf[49]

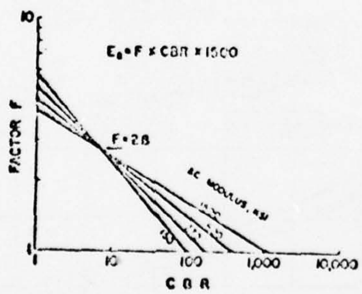

Figure 4.24. Relationship between moduli of subgrade and moduli of granular base (after Dean, Southgate, and Havens[52])

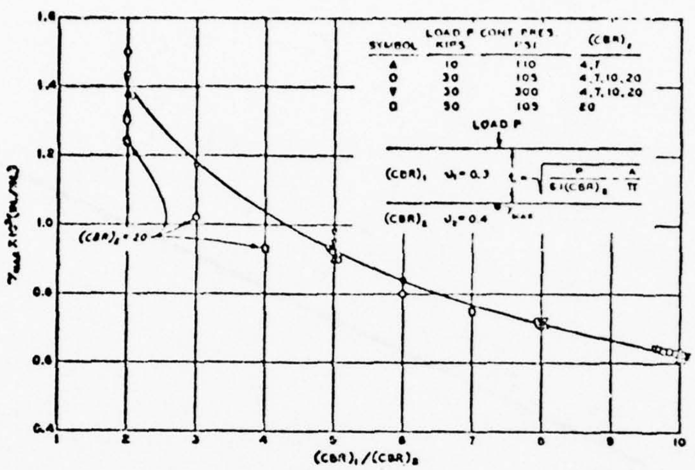

Figure 4.25. Relationship between CBR ratio and computed maximum shearing strain of homogeneous pavements (after Ahlvin, Chou, and Hutchinson[54])

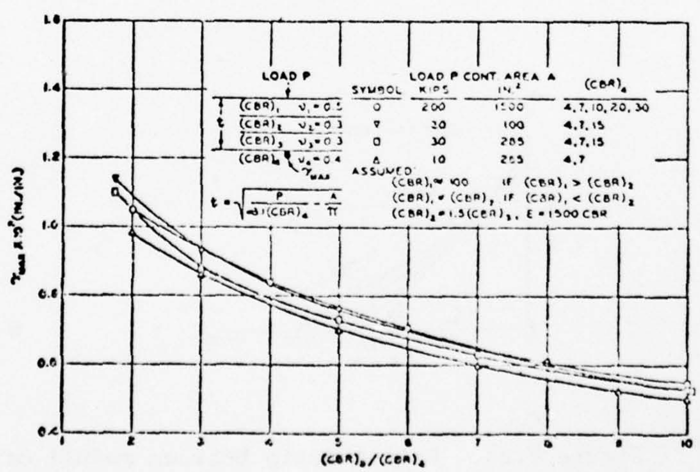

Figure 4.26. Relations between CBR ratio and computed maximum shearing strain for layered pavements (after Ahlvin, Chou, and Hutchinson[54])

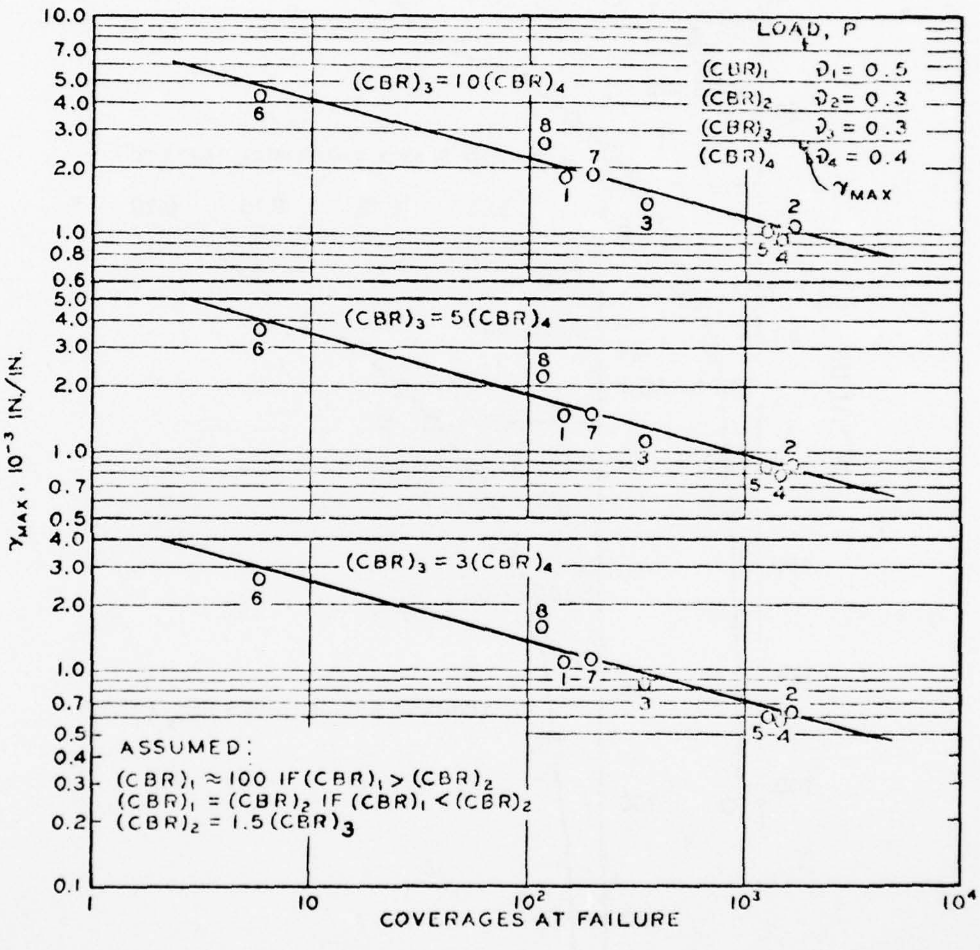

Figure 4.27. Relations between maximum shearing strains and performance for test sections at various CBR ratios (after Ahlvin, Chou, and Hutchinson[54])

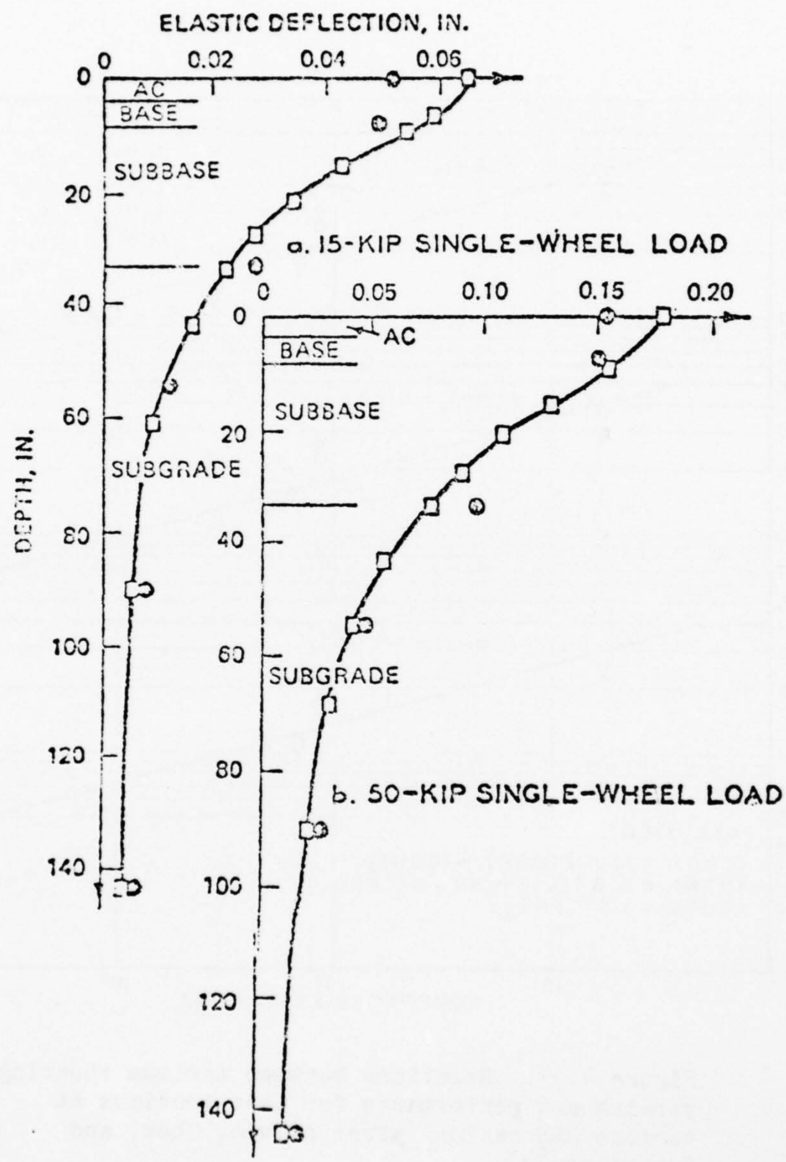

Figure 4.28. Comparisons of measured and computed deflections along the load axis of two single-wheel loads

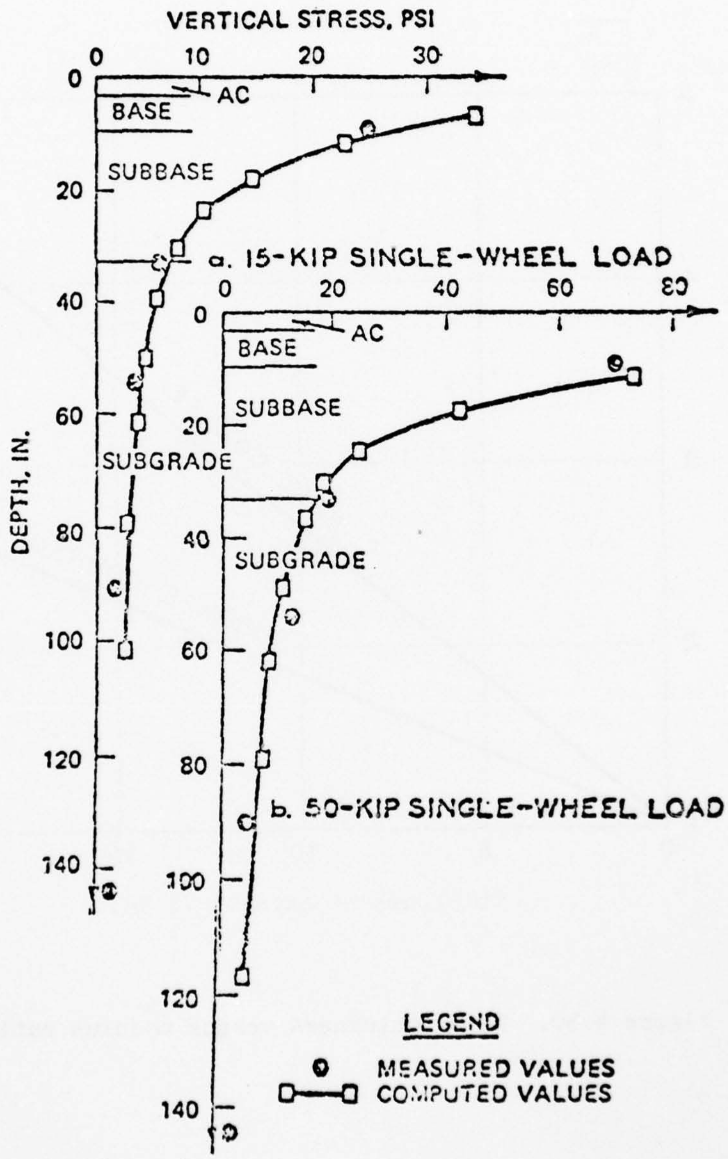

Figure 4.29. Comparisons of measured and computed vertical stresses along the load axis

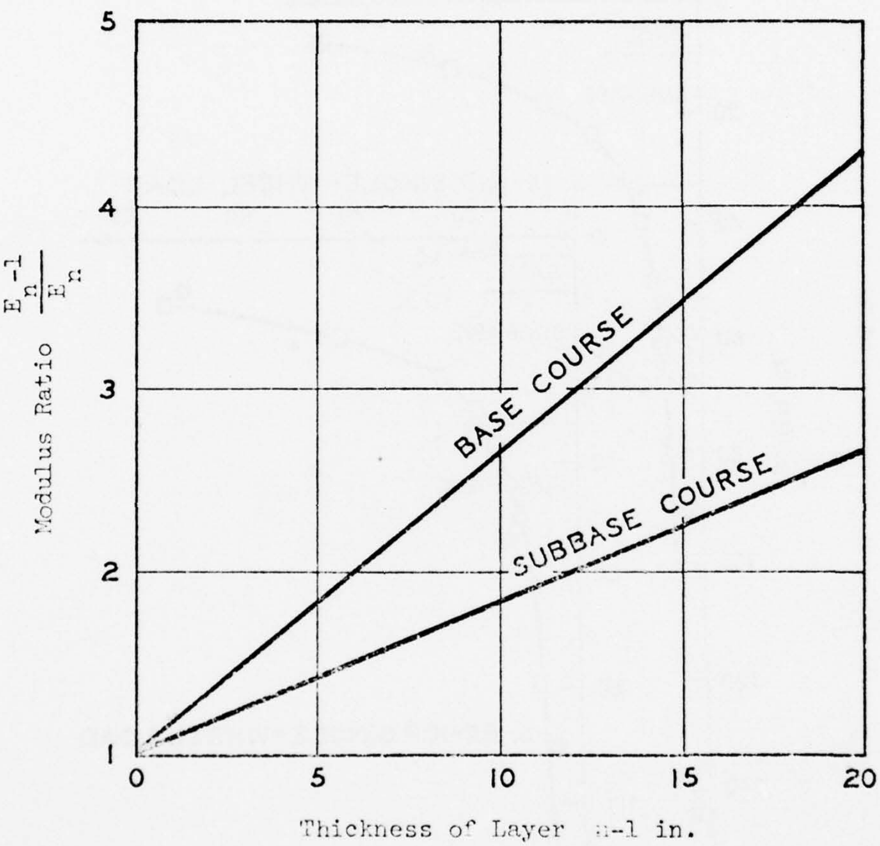

Figure 4.30. Layer thickness versus modulus ratio

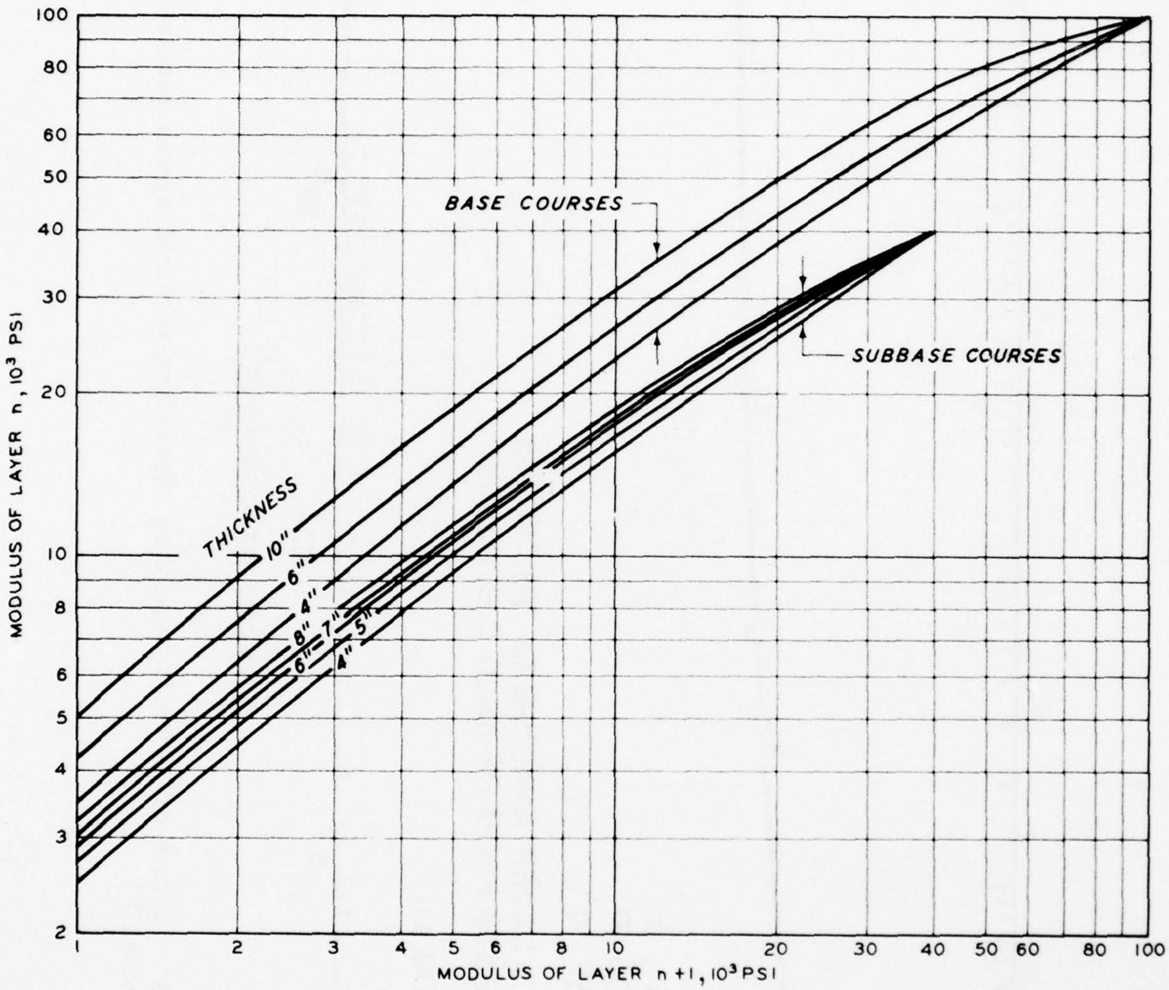

Figure 4.31. Relationships between modulus of layer n and modulus of layer n + 1 for various thicknesses of unbound base course and subbase course

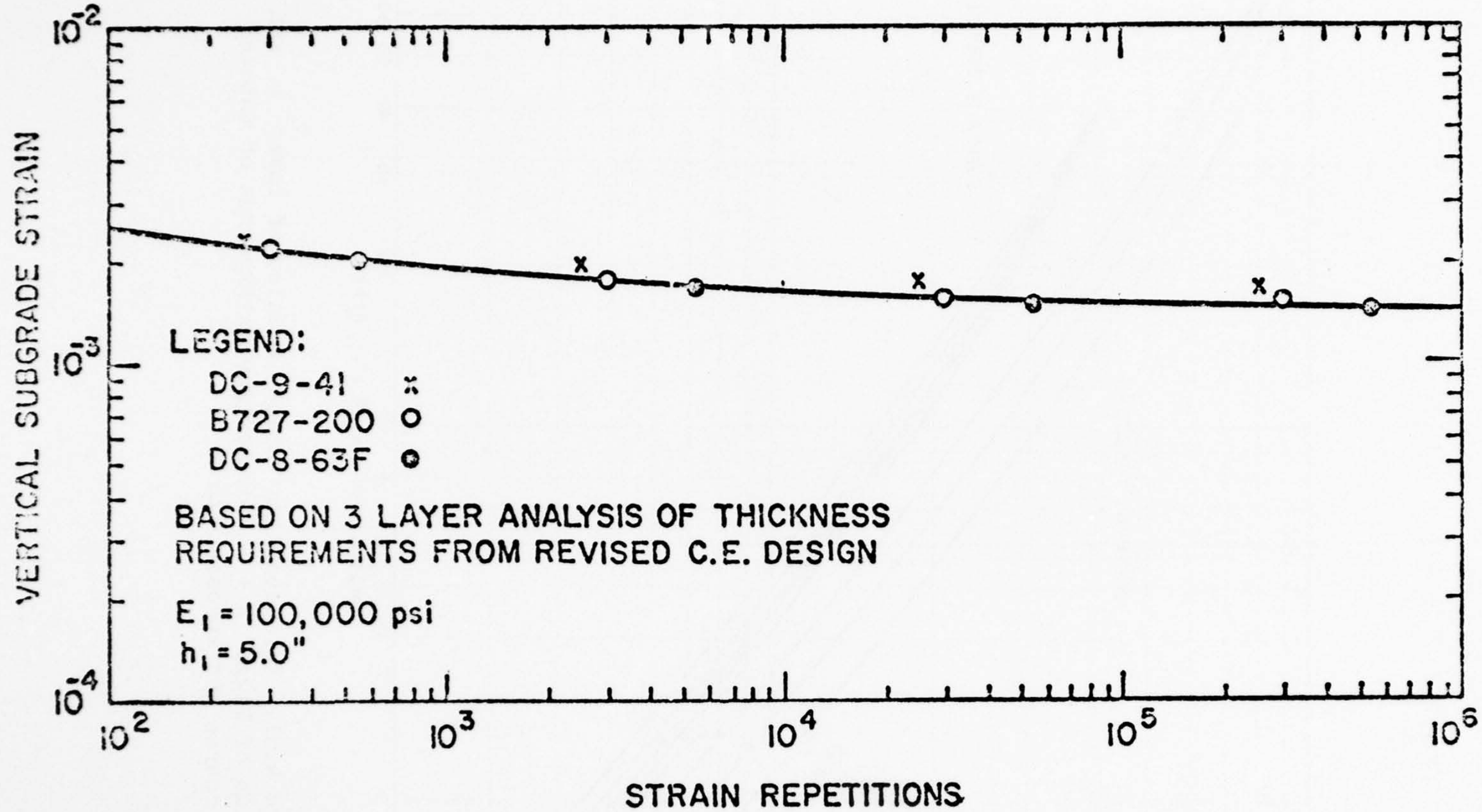

Figure 4.32. Effect of aircraft type upon repetition-vertical strain results (after Witczak[59])

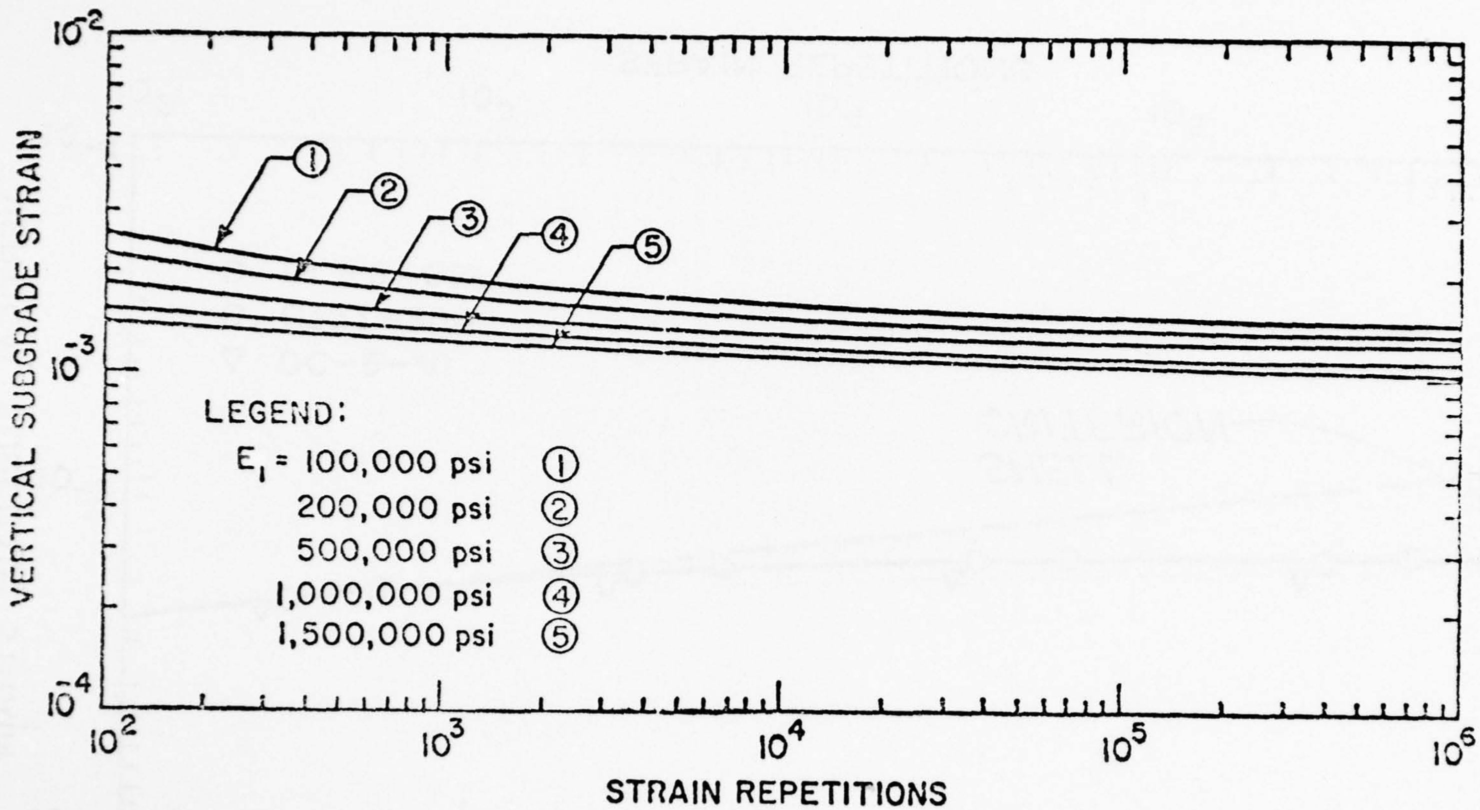

Figure 4.33. Effect of bituminous concrete modulus upon repetition-vertical strain results (after Witczak[59])

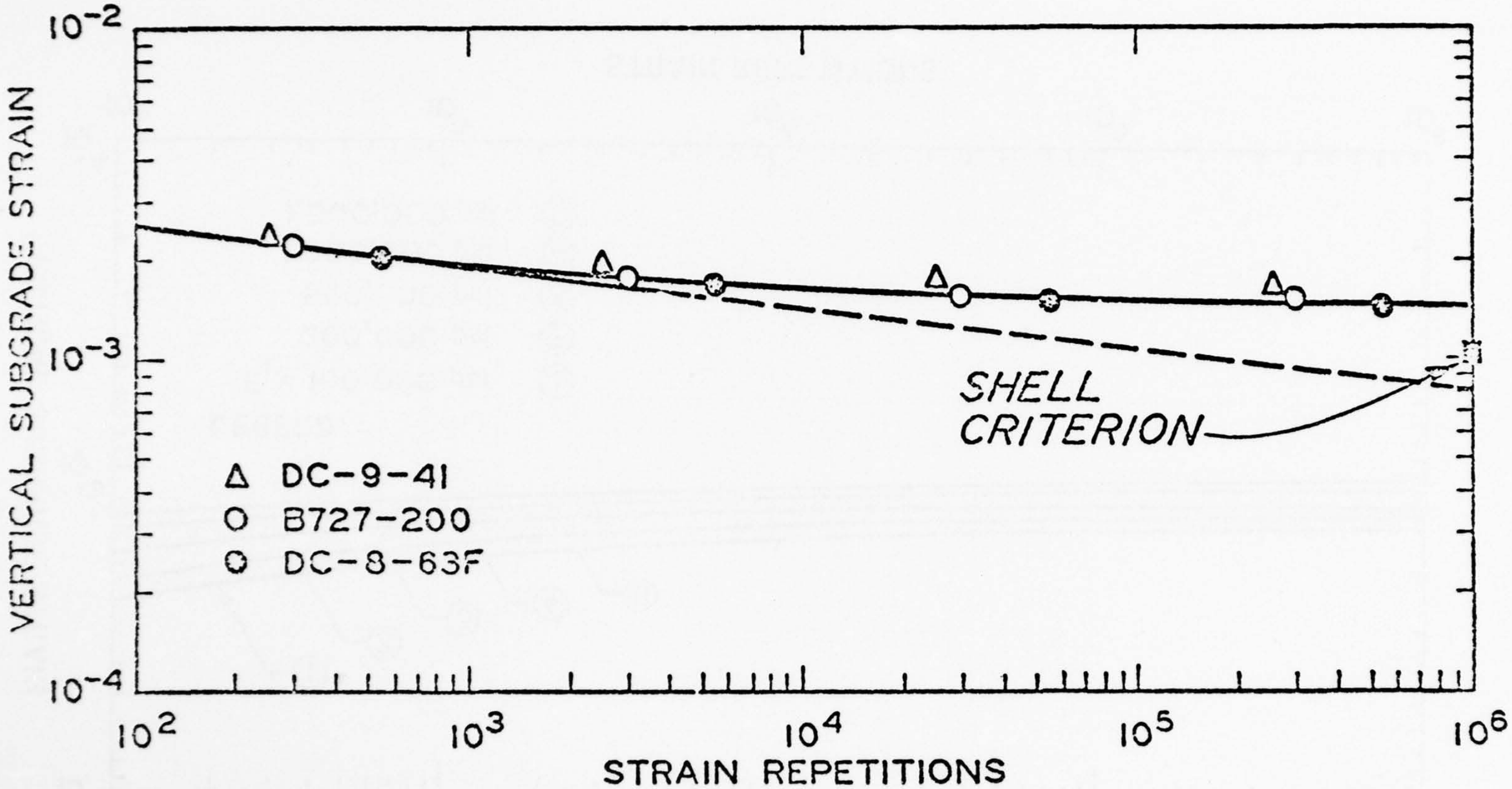

Figure 4.34. Comparisons of failure criteria, permanent deformation mode of distance

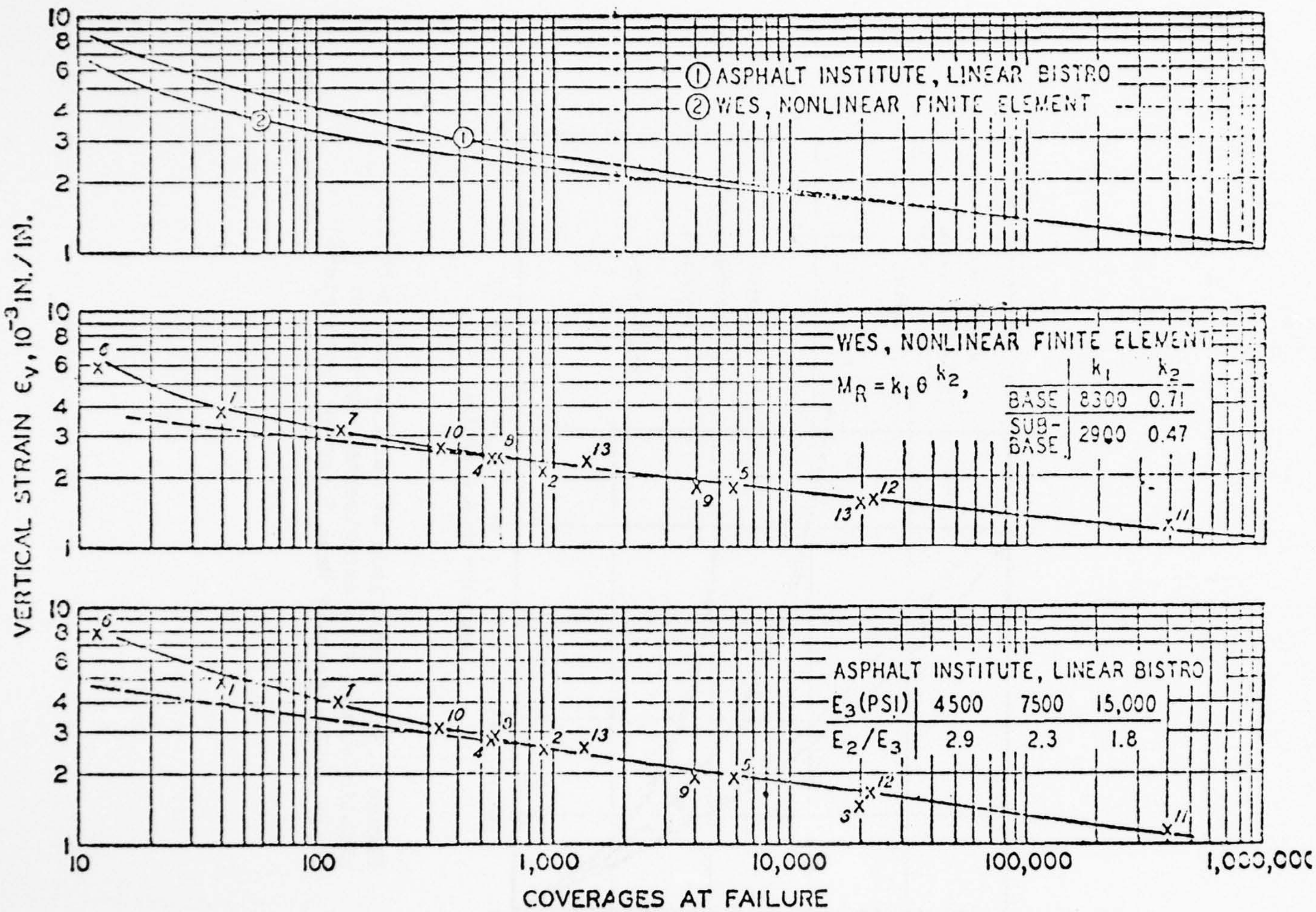

Figure 4.35. Comparisons of moduli of untreated granular materials between The Asphalt Institute and WES

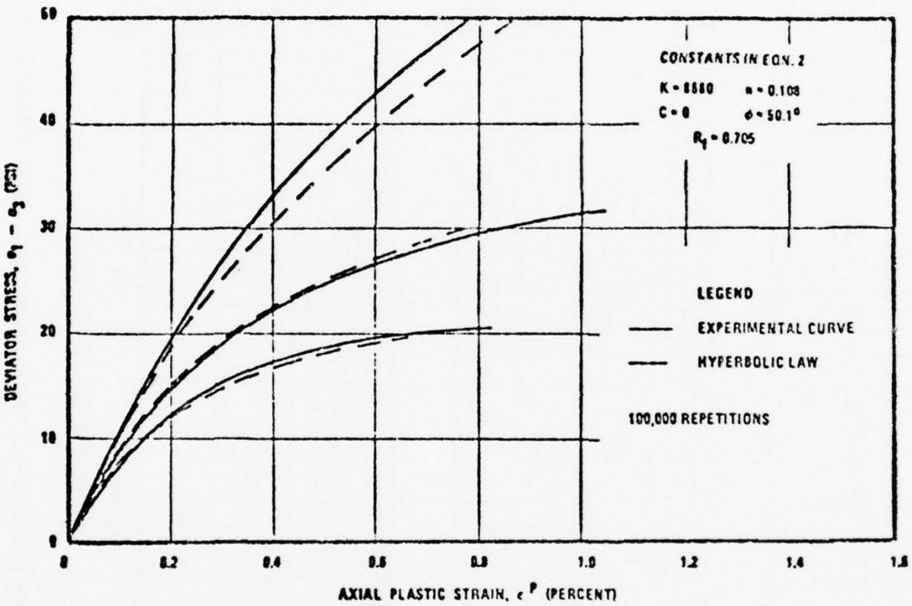

Figure 4.36. Comparison of calculated hyperbolic plastic strain with experimental curves for a 21-79 soil-aggregate Base 5 (after Barksdale[42])

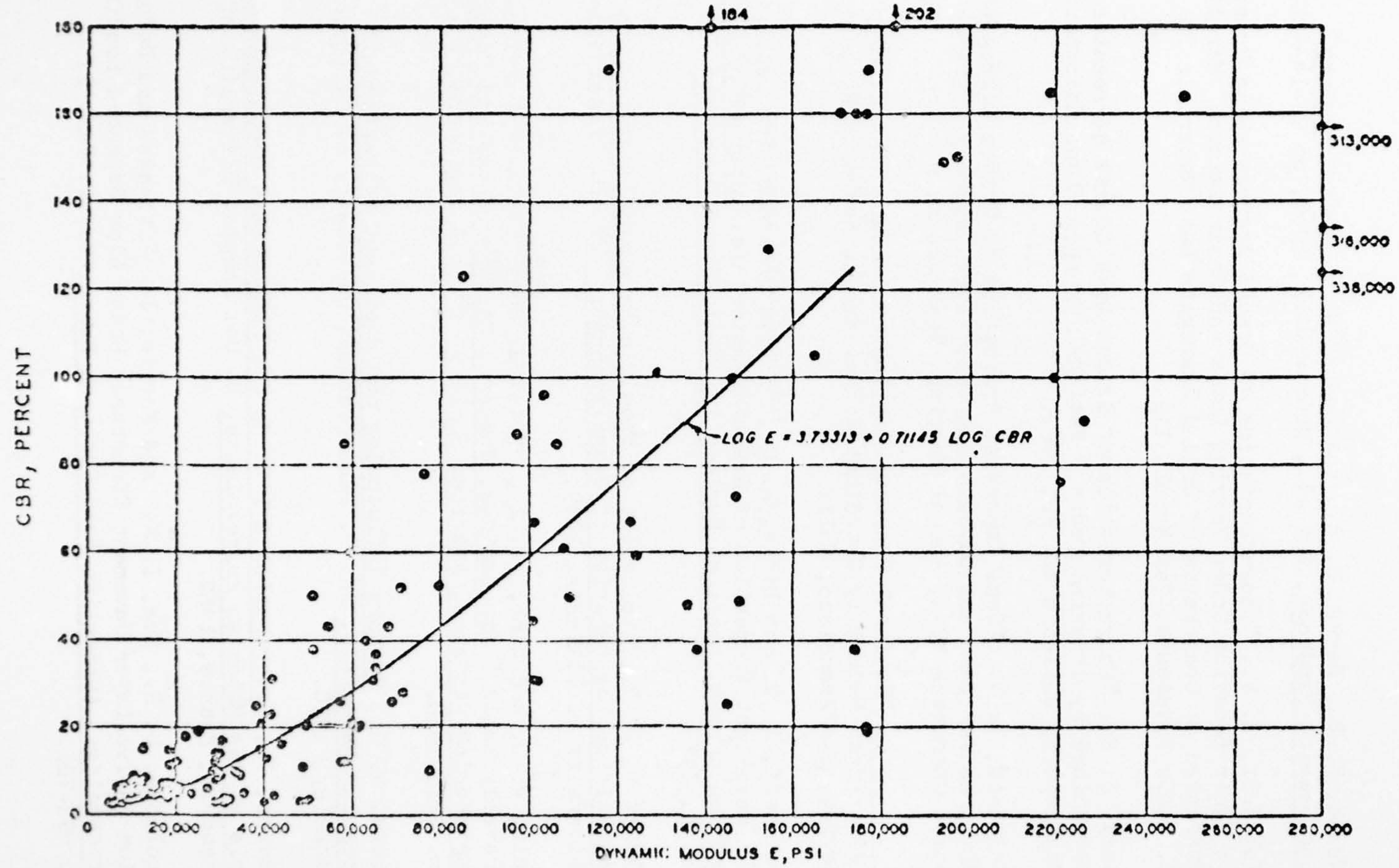

Figure 4.37. An arithmetic plot of dynamic modulus E versus CBR (after Hammitt[80])

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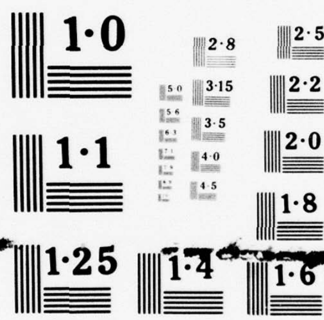
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NOTATION

a	Coefficient
A	Tire contact area
A_1, A_2, A_3	Constants
C	Number of coverages; also, cohesion
CCP	Constant confining pressure
DSM	Dynamic stiffness modulus
e	Void ratio
E	Modulus of elasticity; also, compression modulus; also, dynamic modulus
E_r	Resilient modulus
E_s	Subgrade modulus
f	Frequency
F	Factor
g	Acceleration due to gravity
G	Shear modulus
G_{max}	Maximum shear modulus
K_2	Exponent varying from 0.5 to 0.6; also, modulus of resilient deformation for the unconfined condition
K_3	Constant of proportionality
K_1', K_2'	Material constants
K, K_1, K_2, K_3, K_4	Material constants
LL	Liquid limit
M_r	Resilient modulus
M_R	Resilient modulus
M_z	Modulus of deformation measured in the direction of an applied stress σ_z
n	Constant
N	Number of cycles
P	Wheel load
PI	Plasticity index
q	Effective deviator stress
R_f	Constant relating compressive strength to an asymptotic stress difference

S	Percent saturation
t	Pavement thickness
T	Strain time
V	Wave velocity; also, *initial volume*
VCP	Variable confining pressure
V_c	Compression wave velocity
V_s	Shear wave velocity
w	Water content
w/c	Water content
w_{opt}	Optimum water content
ΔV	Change in volume
$\Delta v/v$	Volumetric strain
γ	Shear strain; also, wet density of soil, pcf
γ_d	Dry density
γ_h	Hyperbolic strain
γ_{max}	Maximum shearing strain
γ_r	Reference strain
ε	Strain
ε^P	Axial plastic strain
ε_a	Axial strain
ε_p	Permanent strain at equilibrium
ε_v	Vertical strain
ε_l	Lateral strain
θ	Static normal stress; also, mean normal stress; also, sum of principal stresses; also, first stress invariant
λ	Wavelength
ν	Poisson's ratio
ν_r	Resilient Poisson's ratio
π	3.1416
ρ	Mass density
σ_d	Deviator stress
σ_m	Mean normal stress
σ_r	Radial stress
σ_z	Applied stress

σ_1	Major principal stress; also, applied vertical stress
σ_3	Confining pressure; also, minor principal stress; also, horizontal stress
σ_θ	Tangential stress
σ'_d	Effective deviator stress
$\bar{\sigma}_3$	Effective confining stress
σ_{oct}	Octahedral normal stress
τ_{oct}	Octahedral shear stress
ϕ	Angle of internal friction

CHAPTER 5: SOIL STABILIZATION

5.1 INTRODUCTION

The purpose of soil stabilization is to improve the strength of marginal soil so that it can be used for subbases and bases. For many years, pavement engineers have experimented with mixtures of soil and cement attempting to produce a low-cost, durable paving material that would utilize native soils which would otherwise be considered to be unsuitable. The properties of a soil may be altered in many ways, among which are chemical, thermal, mechanical, and others. In this chapter, only chemical stabilization is presented.

The various types of stabilization have been categorized according to the properties imparted to the soil. They are summarized in Table 5.1, which is taken from Yoder.[1] Types of admixtures include cementing agents, modifiers, waterproofing agents, water-retaining agents, water-retarding agents, and miscellaneous chemicals. The behavior of each of these admixtures is vastly different from that of the others; each has its particular use, and, conversely, each has its own limitations. In the use of Table 5.1, it may be worthwhile to note that the soil becomes less expansive when the plasticity index (PI) of the soil is reduced. The relationships between the PI and the degree of expansion and the percentage of swell are shown in Table 5.2, which is taken from Packard.[2] The following materials are quoted from Yoder.[1]

> The cementing materials which may be used include Portland cement, lime, a mixture of lime and flyash, and sodium silicate. Portland cement has been used with great success to improve existing gravel roads, as well as to stabilize natural soils. It can be used for base courses and subbases of all types. It can best be used in granular soils, silty soils, and lean clays, but it cannot be used in organic materials. Since soil cement shows strength gains over that of the natural material, it can very often be used for base-course construction, with a light wearing course to resist abrasion.
>
> Another cementing agent, which is used, is hydrated lime. Lime increases soil strength primarily by pozzolanic action, which is the formation of cementitious silicates and aluminates. This material is most efficient when used in granular materials

and lean clays; the quantity required for a proper hydration generally is relatively low. Lime-soil mixtures are generally susceptible to freezing and thawing action; thus, their use is limited to regions of mild climate.

Flyash is produced by burning coal and is generally high in silica and alumina; therefore, the addition of flyash to lime stabilized soils speeds the pozzolanic action. Generally, however, the quantity of flyash required for adequate stabilization is relatively high, restricting its use to areas which have available large quantities of flyash at relatively low cost.

Sodium silicate combined with calcium chloride will cement a soil by formation of a gel. The use of this type of stabilization is restricted generally to deep foundations, since the chemicals must be injected into the soil. Likewise, its use is restricted to sandy materials or other soils which have a relatively high coefficient of permeability.

Many times the use of a cementing material is restricted because of cost, and, therefore, low quantities of the material may be added to the soil merely to modify it. Modifiers which are often used include cement, lime, and bitumen. Cement and lime will change the water film on the soil particles, will modify the clay materials to some extent, and will decrease the soils plasticity index. Small amounts of bituminous materials are very often used in low-grade aggregates, where the function of the bituminous material is to retard moisture sorption of the clay fraction in the soil-aggregate mixture. These modifying materials are generally best adapted to use in borderline base-course materials.

The next category of stabilization includes the water-proofing materials. Foremost among these are bituminous materials which coat the soil or aggregate grains and retard or completely stop sorption of moisture. Bituminous stabilization is best suited for semigranular soils. Retarding or stopping moisture movement into soil can also be accomplished by enveloping the soil in an asphaltic or plastic membrane.

Some chemicals will increase rate of water sorption. They include calcium chloride and sodium chloride. These materials lower the vapor pressure of soil water and lower the freezing point of the soil water as well. Thus, they can be used as a construction expedient to retard evaporation of the soil water during compaction or, in some cases, to prevent freezing of the soil water.

Many other chemicals are available for stabilization. They include compounds which will render a soil hydrophobic. These chemicals will decrease rate of water sorption to a minor extent

but, in general, are very costly, thus limiting their widespread use.

The comparable range of application of various stabilization methods is shown in Table 5.3, which is taken from Ingles and Metcalf.[3] It is seen that reasonable justification exists for such generalizations as "use lime for clays and cement for sands." In Chapter 13 of Reference 3, the exceptional circumstances when special care must be exercised in applying stabilization are described.

In Appendix A to this chapter, a brief description of a soil stabilization index system developed by Texas A&M University for the U. S. Air Force is presented. The system is designed to aid military engineers in selecting the appropriate type and amount of soil stabilizer to use in airfield pavement construction.

5.2 CEMENT STABILIZATION

5.2.1 MECHANISM OF SOIL-CEMENT STABILIZATION

The first noticeable property change that occurs when cement is mixed with moist cohesive soils is a marked reduction in plasticity, probably caused by calcium ions released during the initial cement hydration reactions. As explained in Reference 4, the mechanism is either a cation exchange or a crowding of additional cations onto the clay, both processes acting to change the electrical charge density around the clay particles. Clay particles then become electrically attracted to one another, causing flocculation or aggregation. The aggregated clay behaves like a silt, which has low plasticity or cohesion. Aggregation takes place rather quickly and is caused by the addition of relatively small amounts of cement.

In compacted cement-treated soil, the hydration of the different cement constituents occurs at different rates, providing cementitious amorphous and minutely crystalline hydration products responsible for the characteristic early and long-term strength gains. The cementation is mainly chemical in nature and may be conceptualized as due to the

development of chemical bonds or linkages between adjacent cement grain surfaces and between cement grain surfaces and exposed soil particle surfaces.

With cohesive soils, an important part of the mechanism may be the hardening of clay aggregations by lime liberated as a result of the hydration of the cement. This would explain both the hardened condition of aggregations observed where lumps of stabilized soil are removed from a road base some time after construction and the magnitude of the increase in strength after the hardening of the cement bonds would have been expected to be complete.

The manner in which portland cement stabilizes soils to meet requirements for soil-cement differs somewhat for the two principal types of soils. In fine-grain silty and clayey soils, the cement, on hydration, develops strong linkages among and between the mineral aggregates and the soil aggregates to form a matrix that effectively encases the soil aggregates. The matrix forms a honeycomb type of structure on which the strength of the mixture depends, because the clay aggregations within the matrix have little strength and contribute little to the strength of the soil-cement. The matrix is effective in fixing the particles so they can no longer slide over each other. Thus the cement not only destroys the plasticity but also provides increased shear strength. The surface chemical effect of the cement reduces the water affinity and thus the water-holding capacity of clayey soils. The combination of reduced water affinity and water-holding capacity and a strong matrix provide an encasement of the larger unpulverized raw soil aggregates. Because of its strength and reduced water affinity, this encasement serves not only to protect the aggregates but also to prevent them from swelling and softening from absorption of moisture and from suffering detrimental freeze-thaw effects.

In the more granular soils, the cementing action approaches that in concrete, except that the cement paste does not fill the voids in the aggregate. In sands, the aggregates become cemented only at points of contact. The more densely graded the soil, the smaller the voids, the more numerous and greater the contact areas, and the stronger the

cementing action. Uniformly graded (one-size) sand, which has a minimum of contact area between grains, requires a fairly high cement content for stabilization. Because well-graded granular soils generally also have a low swell potential and low frost susceptibility, it is possible to stabilize them with lesser cement contents than are needed for the uniformly graded sands, the more frost-susceptible silts, and the higher swelling and frost-susceptible clayey soils. For any type of soil, the cementing process is given the maximum opportunity to develop when the mixture is highly compacted at a moisture content that facilitates both the densification of the mix and the hydration of the cement.

Four major variables control the degree of stabilization of soils with cement: (a) the nature of the soil, (b) the proportion of cement in the mix, (c) the moisture content at the time of compaction, and (d) the degree of densification attained in compaction. If the moisture content and the density are controlled in accordance with standard methods (AASHTO T 134 and ASTM D 558) and normal mixing and curing procedures are observed, the nature of the soil and the proportion of cement used determine the degree of stabilization. It is possible, simply by varying the cement content, to produce mixes that, after hydration of the cement, may range from those that result in only a slight modification of the compacted soil (cement-modified soil) to the product known as soil-cement, which must meet certain minimum strength and durability requirements. When moisture is increased sufficiently to produce a plastic mix, and the cement content adjusted to meet strength and durability requirements for the plastic condition, the product becomes plastic soil-cement. The ability to control the properties of the mix to suit the construction and to control the degree of stabilization to satisfy the strength and durability requirements has resulted in the development of these three principal types of cement-treated soil (soil-cement, cement-modified soil, and plastic soil-cement).

5.2.2 FACTORS INFLUENCING PROPERTIES OF CEMENT-STABILIZED SOIL

The effectiveness of soil-cement stabilization is controlled by

many factors and numerous studies have been conducted to investigate their influence on the properties of cement-stabilized soils. For a more detailed review, reference can be made to Highway Research Board Bulletin 292, "Soil Stabilization with Portland Cements," 1961.[4] The major factors affecting the stabilized soil are presented in the following paragraphs.

5.2.2.1 The Soil. The following materials are taken from Ingles and Metcalf.[3] Readers should also refer to Table 5.3 concerning the applicability of cement stabilization to various soil types.

> Any soil, with the exception of highly organic materials, may be treated with cement and will exhibit an improvement in properties; increase in strength, etc. The only practical limits to the range of use of cement stabilization are those imposed by clean, well-graded gravels or crushed rock materials, where stabilization is not only unnecessary but may, in fact, create serious problems of shrinkage cracking; and those imposed by the difficulty of incorporating a (usually) dry fine powder into a (usually) moist heavy clay. Some difficulty has been reported with saline soils but this can be overcome in most cases by increasing the cement content.
>
> A plastic soil may retain some susceptibility to water softening and may swell with increase in moisture content even when stabilized; shrinkage, however, can be a problem with any soil type. For cement stabilization, the upper limit of particle size is about 8 cms (3 in.)--or one-third of the thickness of the compacted layer, but a maximum size of 2 cms (3/4 in.) is to be preferred to give a good surface finish. The lower limit is about 50 percent passing the B.S. No. 200 sieve, with a liquid limit not greater than 50 and a plasticity index not greater than 18.
>
> Soils with a large clay content are difficult to mix and high additive contents are required for an appreciable change in properties. Under laboratory conditions, with elaborate attention to mixing, such heavy clays may be successfully stabilized but, in practice, it is not usual to attempt directly to stabilize with cement a clay soil with a liquid limit greater than about 50.
>
> It is often possible, however, to cement stabilize such heavy clays after pre-treatment (modification), with either cement or, more commonly, hydrated lime. The purpose of the pre-treatment with 2-3 percent of lime or cement is to reduce the plasticity and

render the soil more workable. After curing (compacted or loose) for one to three days, the modified soil is then stabilized with cement in the usual manner.

Cement has been used also to modify the properties of fine crushed rock materials. A very small percentage of cement (1/2 to 1 percent) added to a well-graded, nonplastic, crushed rock material can modify the properties by making the optimum moisture content for compaction less critical and by providing a light "set" or cohesive strength in place of that normally provided by a small amount of plastic fines. Similarly, if a crushed rock has highly plastic fines, the addition of a small amount of cement, perhaps up to 2-3 percent, will reduce this plasticity and produce an acceptable material. In this case, the cement is used for construction expediency rather than for long-term strengthening of the material.

5.2.2.2 Cement Content. For a given soil that reacts normally with cement, the cement content determines the nature of the cement-treated soil. The proportion of cement alters the plasticity, volume change, susceptibility to frost heave, elastic properties, resistance to wet-dry and freeze-thaw alternations, and other properties in different degrees for different soils.

Any cement may be used for stabilization, but ordinarily portland cement is the most widely used. Rapid hardening cements may be useful in organic soils as they provide extra calcium to counteract the presence of the organic matter. In some instances, the use of a retarding agent may assist in overcoming the loss in strength which occurs with ordinary portland cement due to delays in processing and compacting.

One problem in any form of stabilization is that of mixing in an additive, particularly where a dry powder (cement or lime) is to be mixed with a damp soil. It has been suggested, therefore, that a fine cement "diluted" with inert material (ground calcium carbonate, for example) might be equally effective and cheaper than ordinary cements under these conditions. The greater bulk would aid the distribution process so that the same amount of hydratable cement is present throughout the mass. The particle size distribution of ordinary portland cement is fairly well defined between about 0.5 and 100μ, with a mean of about 20μ, but the larger particles never completely hydrate. A $10^{-\mu}$ particle

may take 3 months to hydrate completely and it would seem therefore that a fine cement might be advantageous. A very finely ground cement will produce higher strengths; e.g., a cement finer than the No. 300 sieve will give a 40 percent increase in strength, but such cements are expensive to produce and can be difficult to handle. The suggested stabilizing cement, however, replaces the coarser particles of ordinary cement with an inert filler which maintains the free flowing properties, prevents flash set, and reduces water consumption, shrinkage, and high heat of hydration, and thus combines the benefits of a finely ground cement with the properties of an ordinary cement.

The amount of cement needed in soil stabilization varies with the type of soil. It increases with increasing silt and clay content. The range of the amount of cement needed to meet the criteria established for soil-cement (wet-dry and freeze-thaw tests) is listed in Table 5.4. Generally, for a particular soil, the higher the cement content, the greater the strength of the compacted mixture.

5.2.2.3 Moisture Content. There is no precise measure of the quality of water required, it being generally regarded that potable water is satisfactory. However, highly organic water may cause problems and should be avoided. Waters with a high salt content (sulphates or chlorides in seawater) may be used, provided efflorescence is not likely to be a problem. Most importantly, the quantity of water is related to the amount needed for compaction, not that needed for cement hydration; but in loams and clays, the high water content needed for compaction can result in lower strengths than would otherwise be possible.

Cement-treated soil exhibits the same type of moisture-density relationship as untreated soils. Thus, the moisture content at the time of compaction has a strong influence on the properties of the cement-treated soil. Moisture-density relationships also have a bearing on cement hydration. The significant moisture content is that which prevails at the time of compaction and throughout the curing.

A study was made by Felt[5] to determine the effect of molding moisture content on compressive strength and on loss of density in wet-dry and freeze-thaw tests. Samples were compacted using a constant

compaction effort (standard AASHTO) and varying the moisture content above and below the optimum moisture content, thus also varying the density of the samples. Results indicated that with the compaction effort constant, density varies; the effect of moisture, however, usually overshadows the effect of density. The results may be summarized as: (a) compressive strength increases to a maximum at slightly less than optimum moisture for the sandy soil and the silty soil, and at greater than optimum for the clay soil and (b) results from the wet-dry and freeze-thaw tests show that the clayey soil had less resistance at moisture contents below AASHTO optimum and the silty soil had less resistance in the freeze-thaw test at moisture contents less than optimum. Moisture contents were not so critical in either test for the sandy soil. Felt[5] concluded that, for the best results of strength and durability of soil-cement, the cement, sand mixture should be compacted at optimum moisture content or slightly drier, whereas silty and clayey mixtures should be compacted at moisture contents 1 or 2 percentage points above optimum moisture as determined by the standard AASHTO compaction.

5.2.2.4 Dry Density. The strength and durability of cement-treated soil are strongly influenced by density. The relationship between strength and density approaches a straight line for some soils and cement contents.[6,7] A 5 percent decrease in relative compaction may result in a greater strength reduction than a drop of 10 to 15 percent in cement content (from 10 percent cement to 9 or 8-1/2 percent).[7] Research by the British Road Research Laboratory has established that for a given moisture content the strength of soil-cement is related linearly to the logarithm of the density.[8]

5.2.2.5 Curing Time. Compacted soil-cement mixtures gain strength by cementation processes which continue for months or even years. British studies[8] have indicated that the relationship between the strength of compacted soil-cement samples and the time of curing can be expressed as a straight line on a semilogarithmic or logarithmic plot, depending on the type of soil.

5.2.2.6 Temperature of Curing. The rate of chemical reactions such as hydration, hydrolysis, cementation, and carbonation occurring in compacted soil-cement mixtures depends on the temperature under which these reactions take place. The higher the curing temperature, the faster the rate of reaction. Therefore, the early strength of soil-cement specimens varies with curing temperature. Based on the results of extensive studies, British researchers[9] have noted the following on the influence of temperature on the strength of soil-cement mixtures:

a. The 7-day compressive strength increases with increasing temperature by 2 to 2-1/2 percent per degree Celsius when the temperature is in the vicinity of 25° C (77° F).

b. Soil-cement will harden in cold weather provided the temperature is above 0° C (32° F).

c. If compressive strength is taken as the sole criterion of the quality of soil-cement, less cement is needed in warm weather than in cold weather.

d. Because of ambient temperature differences, soil-cement constructed during warm weather should be 50 to 100 percent stronger than similar construction made during cool weather, at least during the first 3 months of life of the construction.

5.2.2.7 Time Elapsed Between Mixing and Construction. Adding cement to a clay-water mixture immediately increases the pH values of the water and causes flocculation of clay particles. When the time between mixing and compaction becomes too great, it is likely that the large flocculated clay particles will reduce the effectiveness of compaction, owing to the fact that part of the externally applied energy has to be used up for breaking these flocs. West[10] investigated a clay of medium plasticity stabilized with cement, and found by delaying compaction 2-1/2 hours that the moisture-density curve was considerably flattened, the optimum moisture content increased approximately 3 percent, and the maximum density reduced 7 percent.

Other factors such as organic matter, cement type, and addition of additives also significantly influence the mechanical properties of cement-stabilized soil. For laboratory investigations, precautions must be taken to control the test conditions if meaningful results are to be obtained.

5.2.2.8 Deleterious Material. It is generally recognized that organic matter and excess salt content, especially sulphates, can retard or prevent the proper hydration of cement in soil-cement mixtures. One reason why organic matter retards the hydration of cement is because it preferentially absorbs calcium ions and therefore the addition of a ready source of calcium, such as calcium chloride or hydrated lime, may often enable the soil to be treated. This is one case where the use of a special cement, rapid-hardening cement which contains calcium chloride, may be useful. However, the type of organic matter is important and generally an organic content of about 2 percent is regarded as a safe upper limit.

5.2.3 PROPERTIES OF CEMENT-STABILIZED SOIL

5.2.3.1 Compressive and Flexural Strengths. The increase in strength achieved by adding cement to soil is one of the most significant property changes obtained by this treatment. Ranges in unconfined compressive strength of various types of soil-cement are shown in Table 5.5. Soil-cement is more brittle than the untreated compacted soil and has higher values of CBR, plate bearing capacity, and modulus of elasticity.

The broad properties of compacted cement-stabilized soils depend first on cement and secondly on compaction. In the same way as in mechanically stabilized materials, compaction is all-important, and not only in degree but in timing. Compaction after cement hydration is, of course, ineffective. Table 5.6 shows the typical properties of cement-stabilized soils. The properties of a mechanically stabilized soil vary with density and composition. In certain cases, the interrelationships are even more important, because once the cement is hydrated they are irreversible--at least without destruction of the cemented mass.

The strength of a soil-cement mixture depends on many factors. These factors are discussed separately as follows:

5.2.3.1.1 Cement Content. In general, strength increases linearly with cement content, but at different rates for different soils.

Metcalf[11] investigated the effect of cement content on strength for various soils stabilized with ordinary portland cement. The relationships are shown in Figure 5.1. The specimens were cured for 7 days at 25° C at constant moisture content.

5.2.3.1.2 Density. In general, strength increases linearly with density. Figure 5.2 shows such a relationship. A reduction in density of 80 kg/m^3 (5 pcf) may cause a 20 percent reduction in strength, several workers having demonstrated[12,13] a relation between strength and density of the form:

$$S = Ae^{bD} \tag{5.1}$$

where S is the strength, A, b are constants, and D the density; i.e., the logarithm of strength is linearly related to density.

5.2.3.1.3 Delay in Compaction. Delay in compaction allows the hydration process to commence and thus builds up the strength of clods. This is a major cause of loss in strength because the mix becomes more difficult to compact and the final density achieved will therefore be lower. West[10] investigated the loss in strength due to delay in compaction for two soils stabilized with 10 percent cement (Figure 5.3). The soils were a medium clay and a sandy gravel. It can be seen that 75 percent strength can be lost when a time interval of 5 hours exists between mixing and testing. Similar results were also obtained by Wang.[14]

5.2.3.1.4 Curing Time and Curing Temperature. Strength increases gradually with age of curing. Figure 5.4 shows the effect of age on strength of four different soils stabilized with 5 percent cement. Metcalf[12] found that various ways of curing also affect strengths, but the only generalizations that could be made were that higher temperatures increase the rate of gain of strength, and that excessive drying increases strength but provokes cracking.

The influence of curing time on the unconfined compressive (UC) strength of laboratory samples of cement-stabilized Vicksburg silty

clay was investigated in the University of California at Berkeley.[13] The relationships between UC strength and curing time can be mathematically expressed as

$$(UC)_D = (UC)_{D_0} + K \log_{10} \frac{D}{D_0} \tag{5.2}$$

where

$(UC)_D$ = unconfined compressive strength after D days curing

$(UC)_{D_0}$ = unconfined compressive strength after D_0 days curing

K = constant dependent on type of soil and cement content

From results of the laboratory tests, K was found equal to $10C$ for VSC where C is the cement content in percent by weight.

5.2.3.1.5 Comparisons Between Field and Laboratory Sample Strengths. A comparison was made between the unconfined compressive strengths of field undisturbed samples obtained from WES field tests and laboratory mixed and compacted samples at the same water content, dry density, and curing time. It was found that the average ratio of the strength of field samples to that of the samples mixed and compacted in the laboratory was 0.63. Maclean[15] observed that the strength of stabilized soils measured by the mix-in-place method is about two thirds of that made in the laboratory. Wang[14] found a ratio from 0.40 to 0.60 in his experiments. Robert and Schoeneman[16] also found that there is a difference in the strength of soil-cement between laboratory samples and field cores. These results indicate that specimens mixed and compacted in the laboratory are not likely to represent accurately the materials in the field even though the water content and dry density are the same. For the test sections at WES, it was found that the difference in strength between the field undisturbed samples and laboratory samples increased with an increase of cement content. The ratios of the strengths were 0.75, 0.60, and 0.56 for 3 percent, 6 percent, and 10 percent cement contents, respectively.

Strengths were also obtained for field-mixed but laboratory compacted samples. The strengths of these specimens were about 86 percent of those of the samples mixed and compacted in the laboratory at the same

water content and dry density. The percentage also depended on cement content with values of 97 percent, 83 percent, and 78 percent for 3 percent, 6 percent, and 10 percent cement contents, respectively. These values are considerably higher than those for field undisturbed samples. Thus, while the mixing and curing conditions clearly are responsible for a part of the strength difference between field and laboratory samples, a substantial strength difference is attributable simply to the effects of differences in compaction method.

5.2.3.2 Modulus. Many researchers[17-19] have made attempts to define the stress-strain behavior of cement-stabilized soils in compression, in tension, and in flexure, and to produce values for the elastic constants (modulus of elasticity and Poisson's ratio) so that the stresses and strains in pavements may be predicted. Soil-cement, however, is not an elastic material. Metcalf[20] conducted a series of tests on small soil-cement beams in flexure. The clay soil was stabilized with 10 percent cement. The results of Metcalf's study have demonstrated that (a) the stress and strain relations are nonlinear, although the relation is usually linear up to 60 to 70 percent of the failure load as shown in Figure 5.5 and (b) the strain increases gradually under constant creeping load.

Felt and Abrams[18] studied the strength and elastic properties of four compacted soil-cement mixtures. Cement contents ranged from 3 to 18 percent by weight of soil. Compression as well as flexure tests were conducted on samples compacted at ASTM optimum moisture content and maximum density, and a resonant frequency technique was applied to determine the dynamic modulus of elasticity and Poisson's ratio. Values of strength and elastic properties of the four soils studied are listed in Table 5.7. They concluded that, for all soil-cement mixtures at all ages, the modulus of rupture was equal to approximately 20 percent of the compressive strength and that the elastic modulus in flexure (not shown in the table) was equal to the dynamic modulus. However, the static modulus in compression at 33 percent of ultimate load was only equal to approximately 60 to 75 percent of the dynamic modulus.

Morgan and Williams[21] and Parkin, Solly, and Morgan[22] conducted repeated load tests on soil-cement specimens in the laboratory. They found that under repeated triaxial loading, the permanent strain reaches a steady value and that the resilient modulus calculated from the resilient strain after reaching this steady state is essentially constant (it increases slightly with age, presumably as a result of increasing strength of the cement). The stiffness, however, also depends on soil type and increases with confining pressure and with strength; these workers have therefore suggested a nonlinear relation between resilient strain and stress under dynamic conditions. Again, failure occurred under repeated loading at 60 to 70 percent of the static strength. Values of the E (resilient modulus E_r) ranging from 1400 kgf/cm^2 (20,000 psi) to 200,000 kgf/cm^2 (3 million psi), depending on soil type and cement content, have been established. Values of the resilient strain ratio (equivalent to Poisson's ratio ν) range from 0.3 to 0.1.

An extensive study on the resilient properties of soil-cement was conducted in the University of California at Berkeley. The resilient moduli of the specimens were determined in compression M_{Rc} and in flexure M_{Rf}. The following materials were taken from Mitchell, Jeng, and Monismith.[13]

5.2.3.3 Resilient Modulus in Compression. A typical relationship for the resilient modulus in compression and the UC strength is shown in Figure 5.6. Resilient moduli depend on the number of load repetitions, but become essentially constant after between 300 and 10,000 repetitions, depending on both material and loading conditions. The following empirical expressions are based on constant moduli. The relationship may be represented mathematically using the following notation:

UC = unconfined compressive strength, psi

$g(\sigma_d)$ = function of deviator stress

σ_3 = confining pressure, psi

σ_1 = major principal stress, psi

I_1 = first stress invariant, psi

K_c = constant dependent on material

For Vicksburg silty clay stabilized with 13 percent cement and having a confining pressure of zero, Shen[23] states that

$$M_{Rc} = 0.02(10)^{-0.0278\sigma_d}(UC)^{3.5} \tag{5.3a}$$

or

$$M_{Rc} = 0.025(\sigma_d)^{-1.2}(UC)^{3.5} \tag{5.3b}$$

For Elliot sand mixture with 7 percent cement and a confining pressure of zero, Shen[23] states that

$$M_{Rc} = K_c g(\sigma_s d)(UC)^{2.2} \tag{5.4}$$

For Vicksburg silty clay with 3 percent cement, Mitchell, Fossberg, and Monismith[24] have stated that

$$M_{Rc} = 13.0(\sigma_d)^{-0.54}(\sigma_3)^{0.6}(UC)^{1.65}\ , \ \sigma_3 = 0 \tag{5.5a}$$

or

$$M_{Rc} = 2.3(\sigma_1)^{-2.56}(I_1)^{2.62}(UC)^{1.65} \tag{5.5b}$$

For buckshot clay with 6 percent cement and a confining pressure of zero, Mitchell, Fossberg, and Monismith state that

$$M_{Rc} = K_c g(\sigma_d)(UC)^{1.97} \tag{5.6}$$

For Richmond Field Station silty clay with 3 percent cement, Wang[14] states that

$$M_{Rc} = 190(\sigma_d)^{-0.436}(\sigma_3)^{0.272}(UC)^{1.7}\ , \ \sigma_3 = 0 \tag{5.7a}$$

or

$$M_{Rc} = 135(10)^{0.69\sigma_3}(\sigma_d)^{-0.436}(UC)^{1.7} \qquad (5.7b)$$

For Richmond Field Station silty clay with 6 percent cement

$$M_{Rc} = 152(\sigma_d)^{-0.236}(\sigma_3)^{0.695}(UC)^{1.88} \ , \ \sigma_3 = 0 \qquad (5.8a)$$

or

$$M_{Rc} = 100(10)^{2.19\sigma_3}(\sigma_d)^{-0.236}(UC)^{1.88} \qquad (5.8b)$$

For Richmond Field Station subgrade clay and a confining pressure of 2 psi, Wang[14] and Fossberg[25] state that

$$M_{Rc} = 600(\sigma_d)^{-0.55}(UC)^{0.88} \qquad (5.9)$$

From the available data it may be deduced that

$$M_{Rc} = K_c(\sigma_d)^{-K_1}(\sigma_3)^{K_2}(UC)^n \qquad (5.10)$$

where

K_1 = 0.2 to 0.6

K_2 = 0.25 to 0.7

$n \approx 1.0 + 0.18C$, where C is the cement content by weight

5.2.3.4 Resilient Modulus in Flexure. A typical relationship is shown in Figure 5.7. For Vicksburg silty clay with 13 percent cement, Shen[23] states that

$$M_{Rf} = 3.2 \times 10^5(10)^{0.00018UC} \qquad (5.11)$$

For Elliot sand mixture with 7 percent cement, Shen[23] states that

$$M_{Rf} = 6.5 \times 10^5(10)^{0.0008UC} \qquad (5.12)$$

For Vicksburg silty clay with 3 percent cement, Mitchell and Monismith[26] state that

$$M_{Rf} = 9.0 \times 10^{3}(10)^{0.0132UC} \tag{5.13}$$

For Richmond Field Station silty clay, Wang[14] states that, for 3 percent cement content,

$$M_{Rf} = 2.0 \times 10^{4}(10)^{0.009UC} \tag{5.14}$$

and for 6 percent cement content,

$$M_{Rf} = 5.6 \times 10^{4}(10)^{0.0035UC} \tag{5.15}$$

According to the conclusions of previous investigations, the resilient modulus in flexure is independent of applied flexural stress intensity. The effect of confining pressure has never been studied. For the present, it will be assumed that confining pressure exerts no influence on the resilient modulus in flexure. It is concluded from the available information that the resilient modulus in flexure can be expressed as

$$M_{Rf} = K_{F}(10)^{m(UC)} \tag{5.16}$$

where

K_f = constant dependent on material
$m = 0.04(10)^{-0.186C}$
C = cement content

Anisotropic pavement properties have been found by some investigators (e.g. Fossberg[25]), and the modulus of resilient deformation is usually different in compression than in flexure. The results of beam tests under repeated loading and findings by other investigators have shown, however, that the theory of elasticity is valid in flexural beam tests for resilient deformations. Furthermore, the flexure test would appear more

representative for pavement systems than the direct tension test. Thus, it would seem reasonable for pavement systems to use the resilient properties in the vertical and horizontal directions as determined by compression tests and flexural beam tests, respectively.

5.2.4 FATIGUE PROPERTIES

Fatigue is one of the major causes of pavement failure. Bofinger[27,28] studied the fatigue behavior of three heavy clays stabilized with different amounts of cement. He concluded that:

a. Under compressive repeated loads, no fatigue failure occurred for stress levels up to 95 percent of the compressive strength.

b. The fatigue life is shorter under direct tensile stresses than under flexural stresses.

c. The tensile fatigue curves for cement-stabilized clays are dependent on cement content.

d. The fatigue strength is reduced by soaking of the samples.

In one of his studies, Bofinger[27] found a tensile fatigue limit for soil-cement, while in another study[28] he concluded that there was no true tensile fatigue limit for soil-cement according to the best fitting curves selected by the least squares method for the test data.

Larsen and Nussbaum[29] concluded that the fatigue behavior of soil-cement can be expressed in the form

$$\frac{R_c}{R} = aN^{-b} \tag{5.17}$$

where

R_c = critical radius of curvature, i.e., the minimum curvature at failure under static load

R = allowable radius of curvature for a given number of load repetitions

a = factor varying with thickness of pavement h

N = number of load repetitions

b = constant dependent on type of soil

In a later paper of Larsen, Nussbaum, and Colley[30] the exponent b in Equation 5.17 was given as 0.025 for granular soils and 0.050 for fine-grained soils, and a was expressed by

$$a = \frac{h^{3/2}}{2.1h - 1} \tag{5.18}$$

where h is pavement thickness. They found that the subgrade strength had no significant influence on the fatigue characteristics.

Some fatigue test results for Vicksburg silty clay stabilized with 3 percent cement tested after a 24-hour curing period were obtained by Mitchell and Monismith.[26] These results can be expressed in the form

$$S = aN^{-b} \tag{5.19}$$

where S is the stress level; i.e., ratio of flexural stress causing failure at N load repetitions to the flexural strength of the material.

Pretorius[31] studied the fatigue behavior of soil-cement prepared from a mixture of gravel and Richmond Field Station silty clay. The results were

$$\log N_f = 9.11 - 0.0578\varepsilon_i, \text{ or } N_f = \left(\frac{142}{\varepsilon_i}\right)^{20.3} \tag{5.20a}$$

$$\log N_f = 7.481 - 0.0162\sigma_i \tag{5.20b}$$

$$\log N_f = 10.281 - 11.28 \frac{R_c}{R}, \text{ or } \frac{R_c}{R} = 0.814N_f^{-0.037} \tag{5.20c}$$

where

N_f = number of load repetitions at failure

ε_i = initial flexural strain based on strain gage measurements

σ_i = initial flexural stress

The data were plotted according to stress level, giving

$$S = 0.91N_f^{-0.048} \tag{5.21}$$

Because the data are limited, only the following tentative conclusions can be drawn:

a. The fatigue behavior of cement-treated soils can be expressed as

$$S = aN^{-b} \qquad (5.22)$$

where

S = flexural stress level

$a \simeq 0.95$

$b \approx 0.03$ to 0.06

b. No fatigue failure will occur for flexural stress levels below about 50 percent of the flexural strength of the material.

For convenience, stress was used as the controlled factor to describe fatigue.

5.2.5 TENSILE STRENGTH

Pavement failure many times is caused by tensile stresses on the underside of the slab under the load. The tensile strength of soil-cement is therefore of importance. Metcalf[32] found the tensile strength at the optimum moisture content and maximum density were generally about 10 percent of the compressive strength. Figure 5.8 shows such a relationship.

5.2.6 INTERRELATIONSHIP BETWEEN STRENGTHS EVALUATED BY DIFFERENT TESTS

The following materials are taken from Mitchell, Jeng, and Monismith.[13]

5.2.6.1 Strength in Unconfined Compression. The results of undrained triaxial tests on undisturbed samples and laboratory compacted samples were correlated with UC strength values[13] for a Vicksburg silty clay stabilized with 3, 6, and 10 percent cement. It was found that the cohesion intercept c of the Mohr envelope increases with an increase in UC strength. This relationship can be expressed by

$$c = 7.0 + 0.0225(UC) \qquad (5.23)$$

This relationship is quite similar to that found by Thompson[33] for lime-treated soils. The friction angle appears to depend on cement content only. This finding is supported by the results obtained by Balmer[19] and Wissa and Ladd.[34] This fact can be explained, as suggested by Wissa and Ladd, on the basis that the increase of internal friction angle is due mainly to the formation of cemented aggregates, and the formation of cemented aggregates depends on the cement content.

5.2.6.2 Flexural Strength. A linear relationship exists between UC strength and flexural strength for cement-stabilized Vicksburg silty clay. The cement contents were 3, 6, and 10 percent. The flexural strength is about 25 to 35 percent of the compressive strength, slightly higher than the ratios obtained by Felt and Abrams,[18] Barenberg (25 percent),[35] and Wang (20 to 35 percent).[14] Thompson[33] found that the flexural strength is about 25 percent of UC strength for lime-treated soils. The linearity of the relationship indicates that the modulus of rupture can be estimated in terms of the compressive strength for a given stabilized soil. It is interesting to note that the data suggest a UC strength of about 25 psi at zero flexural strength. This corresponds to untreated soil, which possesses a compressive strength but no tensile strength.

5.2.6.3 California Bearing Ratio (CBR). A relationship was found between laboratory CBR values and laboratory UC strength for the materials used in the test sections at WES (Figure 5.9). It is not linear as found by Maclean,[15] but it can be approximated as a bilinear relationship. Maclean's data, except those for sand-cement, agree well with this relationship.

The shape of the curve in Figure 5.9 can be explained by the failure mechanism in the CBR test. For a low-strength soil, the penetration of the CBR piston mainly causes punching failure in which only local failure in the soil occurs. At higher strengths, the penetration causes general failure in the soil under the piston, and the full strength is mobilized. Because the punching effect and local failure behavior are different for different types of soils, the curve may not be the same

for all soils. In the range of values in practice, a linear relation between log CBR value and log UC strength seems appropriate.

5.2.7 DESIGN APPLICATIONS

In the United States, the cement requirement for stabilization of a given soil usually is determined by a series of wet-dry and freeze-thaw tests on compacted specimens. The cement percentage is selected by comparing the weight losses during the resistance tests with the allowable loss. In Great Britain, the cement requirement is determined on the basis of compressive strength. Specifications usually require a field compressive strength of 250 psi. It has been found that normal construction methods result in a field strength equal to about 60 percent of the laboratory strength for a given cement treatment. Therefore, the cement content is determined as that necessary to give a laboratory compressive strength equal to 42.0 psi.

A detailed literature review on the design of soil-cement bases and pavements and performance data of field tests conducted in the United States and overseas was presented by Mitchell and Freitag[36] in 1959.

An extensive series of field tests was conducted at WES. The pavements were constructed using Vicksburg silty clay treated with cement in amounts of 3 to 10 percent. The subgrade was a heavy clay with CBR values in the range of 2.7 to 13. The traffic was 10,000- to 50,000-lb single-wheel loads with aircraft-type tires. Wearing surfaces were not placed on the pavements. The traffic test data were analyzed by faculty members of the University of California at Berkeley.[13] The following are conclusions of the analysis: (a) the UC strength can be used to correlate different cement-stabilized soil properties; (b) performance correlated well with cement content, pavement thickness, subgrade strength, pavement strength, and traffic; (c) the performance data enable development of design curves for pavements containing cement-stabilized soil layers; and (d) prediction of stresses, deformations, and fatigue behavior in cement-stabilized soil pavements is possible using the finite element method and elastic layer theory. It was hoped that the

results of these and related studies could be combined to develop improved design methods for pavements containing cement-stabilized soil layers. A framework for the design of cement-stabilized layers utilizing the results of the investigations obtained from the studies contracted with WES as well as the results of studies of cement-stabilized materials completed by other investigators was reported by Mitchell, Dzwilewski, and Monismith[37] of the University of California at Berkeley.

In recent years, cement-treated subbases have been used frequently in rigid pavements. The performance of pavements carrying heavy loads and high volumes of traffic has shown the benefits of stabilized subbases.[2] Cement-treated subbases offer many benefits in addition to the prevention of mud pumping:

<u>a</u>. Provide impermeable, uniform, and strong support for the pavement. The impermeable layers reduce the amount of surface water reaching the subgrade and eliminate the possibility of excessive pore pressures that otherwise could develop in granular subbases.

<u>b</u>. Eliminate subbase consolidation.

<u>c</u>. Greatly improve load transfer at joints.

<u>d</u>. Expedite construction because the stable working base eliminates shutdowns due to adverse weather conditions.

<u>e</u>. Provide firm support for the slipform paver or side forms, thus contributing to the construction of smoother pavements.

Cement content for cement-treated subbases for airport pavements is determined by standard laboratory wet-dry and freeze-thaw tests (ASTM D 559 and D 560) and PCA weight-loss criteria.[38] Other procedures that give an equivalent quality of material may be used. Details of material requirements and construction methods for quality cement-treated subbases are given in References 38-40.

5.3 LIME AND FLY ASH STABILIZATION

5.3.1 MECHANISMS OF SOIL-LIME STABILIZATION

Lime reacts with the clay minerals of the soil, or with any other fine, pozzolanic component such as hydrous silica, to form a tough water-insoluble gel of calcium silicate, which cements the soil particles. The cementing agent is thus exactly the same as for ordinary portland cement,

the difference being that with the latter the calcium silicate gel is formed from hydration of anhydrous calcium silicate (cement), whereas with the lime the gel is formed only after attack on and removal of silica from the clay minerals of the soil. The contrast with cement stabilization is that the latter is essentially independent of soil type, while the rate for lime-stabilized soils is different for each soil type.

Thompson[41] has given a brief description of basic reactions which contribute to the improvements in engineering characteristics of lime-soil mixtures. This description is presented below.

5.3.1.1 Cation Exchange. The general order of replaceability of the common cations associated with soils is given by the lytropic series, $Na^{+} < K^{+} < Ca^{++} < Ng^{++}$. Any cation will tend to replace the cations to the left of it, and monovalent cations are usually replaceable by multivalent cations. The addition of lime to a soil supplies an excess of Ca^{++} and cation exchange will occur, with Ca^{++} replacing dissimilar cations from the exchange complex of the soil. In some cases the exchange complex is practially Ca^{++} saturated before the lime addition and cation exchange does not take place, or is minimized.

5.3.1.2 Flocculation and Agglomeration. The addition of lime to a fine-grained soil causes flocculation and agglomeration of the clay fraction. These reactions result in an apparent change in texture, the clay particles "clumping" together into larger sized "aggregates." The flocculation and agglomeration is affected by the increased electrolyte content of the pore water and also as a result of ion exchange by the clay to the calcium form.

The influence of cation exchange, flocculation, and agglomeration on the plasticity and shrinkage properties of lime-soil mixtures were studied by Thompson. The study indicated that these reactions are primarily responsible for the changes in plasticity, shrinkage, and workability characteristics of lime-soil mixtures. These beneficial changes were noted for all soils studied and relatively small percentages of lime were required to achieve the changes.

Thompson reported that cation exchange, flocculation, and agglomeration are not the basic lime-soil reactions which are responsible for the

marked strength increases noted for many lime-soil mixtures.

5.3.1.3 Lime Carbonation. Lime reacts with carbon dioxide to form the relatively weak cementing agents calcium and magnesium carbonate, depending on the type of lime used. When lime-treated soils were laboratory-cured in the open air, calcium carbonate forms; it is a condition conducive to promoting carbonation.

5.3.1.4 Pozzolanic Reaction. The pozzolanic reaction referred to in lime-soil stabilization literature is a reaction between soil silica and/or alumina and lime to form various types of cementing agents. These cementing agents are generally regarded as the major source of the strength increases noted in lime-soil mixtures. Possible sources of silica and alumina in typical soils include clay minerals, quartz, feldspar, micas, and other similar silicate or aluminosilicate minerals.

5.3.2 PROPERTIES OF LIME-STABILIZED SOIL

The properties of lime-stabilized soils vary in a similar manner to those found for cement-stabilized soils. The differences lie mainly in the effect of additive content, the effect of time, and the effect of temperature. The UC strength of soil-lime mixtures increases with increasing lime content to a certain level, usually about 8 percent for clay soils. The rate of increase then diminishes until no further strength gain occurs with increasing lime content (in contrast to cement stabilization where the increase in strength continues to quite high cement contents (20 percent)). Because with lime-soil mixtures there is no rapid cementation akin to the setting of concrete, the effect of delay in compaction is far less important with lime stabilization. Because there is, in general, no urgency for compaction, the process of lime stabilization is more flexible in the field. The gain in strength with time of a compacted soil-lime mixture broadly follows the pattern for soil-cement mixtures but the effect of temperature is more marked.

The lime-induced cation exchange, flocculation, and agglomeration and pozzolanic reactions have been found to improve the engineering properties of fine-grained soils. Cation exchange and flocculation and agglomeration reactions occur with almost all fine-grained soils when

treated with lime. Both reactions are immediate. Lime-stabilized soils exhibit a substantial reduction in plasticity and an apparent increase in grain size. These combined effects result in improved workability characteristics of lime-stabilized soils. In addition, the lime-stabilized soil is less susceptible to swell, more moisture-resistant, and possesses improved stability as compared to the same untreated natural soil. Increased stability has been noted by investigators in the form of increased CBR strength of lime-soil mixtures as compared to the CBR of the untreated soils. In conjunction with field tests to evaluate the benefits of stabilizing a lean clay with quicklime, the Corps of Engineers[42] found that within 1 day the CBR strength went from 3.0 for the natural soil to 53 with only a 4 percent lime treatment of the same soil. Performance tests in conjunction with this study showed that a lime-stabilized 16-in. layer of the lean clay overlying a subgrade with a CBR of 4 was able to sustain 2000 coverages of a 10,000-lb single-wheel load with 100-psi tire pressure. The same wheel load on the untreated subgrade became immobilized during the first pass. Loading of the lime-treated section was begun 1 day after construction; therefore, the immediate effects of lime-soil stabilization appeared to be the primary cause of the increased stability.

The use of the immediate effects of lime-soil stabilization in the construction of Interstate I-180 near Princeton, Illinois, was reported by Thompson.[43] The subgrade was a loess-derived soil with an A-6 AASHTO classification. Construction was literally at a standstill during the early spring months due to the very unstable condition of the wet subgrade. It was decided to attempt a new technique of deep plowing a 5 percent treatment of lime to a depth of 24 in. As a result of the immediate effects of the lime-soil treatment, the subgrade's plasticity was reduced, the workability improved, and the overall stability was greatly enhanced. Following compaction, the lime-stabilized subgrade was stable to the point of easily supporting loaded trucks.

In considering the various factors that affect the reaction of a soil to lime, Thompson[43] has determined for Midwestern United States soils that organic carbon content, clay content, clay mineralogy, pH of the

soil, natural drainage, soil horizons, and calcium carbonate equivalents are significant. High organic carbon in a soil retards the lime-soil pozzolanic reaction. A minimum clay content of 10 to 12 percent is necessary for lime to react with a soil to develop a substantial strength increase. All clay minerals are potentially "reactive" but montmorillonite and mixed-layer clay minerals are more reactive than the other clay minerals. High soil pH indicates a potential "reactive" condition. Poorly drained soils are generally "reactive," whereas well drained soils are generally "nonreactive." The A horizons of soil profiles are "nonreactive." The B horizons vary from "reactive" to "nonreactive." All calcareous soils are "reactive."

A knowledge of the factors which influence the lime reactivity of soils is an important facet of the study of lime-soil stabilization, but perhaps not as important as an understanding of the methods for evaluating and the actual use of lime-soil mixtures as engineering materials. Studies at the University of Illinois[44,45] have considered the engineering properties of typical cured lime-soil mixtures. As a result of these studies, the following conclusions were reached regarding the shear strength and elastic properties:

a. Lime stabilization of "reactive" soils results in a substantial increase of shear strength of the lime-soil mixture over that of the natural soil.

b. The modulus of elasticity E of the lime-soil mixture is much larger than the E of the untreated soil.

The effect of the application as hydrated lime slurry and hydrated lime powder was examined by Davidson, Noguero, and Sheeler[46] who showed the two methods to be equally effective. The slurry concentration was 50 percent. Where water is to be added for compaction, slurry application has the great advantage of being dust free and more likely to give a uniform distribution of the lime.

The use of quicklime (calcium oxide) to stabilize fine sands and loessial soils is common in Germany[47] and has been studied by Laguros et al.[48] who showed quicklime to be better than hydrated lime (calcium hydroxide). Quicklime takes up water to the amount of one third its own

weight to hydrate, and thus, in addition to the "apparent" drying out caused by the increase in plastic limit, there is an actual drying out by the removal of water to form the hydrated lime and by the heat produced by hydration.

Brand and Schoenberg[49] reported their experiences working with loess. They found that 3 percent quicklime can transform a soil from, for example, the liquid condition, with a field moisture content of 21 percent and a liquid limit of 23 percent, to the plastic condition, with a field moisture content reduced to 16 percent and the plastic limit increased from 18 to 21 percent.

In conclusion, lime changes the physical characteristics of most clay soils in varying degrees. The National Lime Association[50] has presented the following summary of the effects:

a. The plasticity index drops sharply--more than fourfold in some instances. This is generally due to the liquid limit decreasing and the plastic limit increasing.

b. The soil is agglomerated, decreasing the soil binder content (minus No. 40 mesh particles) substantially.

c. Lime (and water) accelerates the disintegration (breaking up) of clay clods during mixing. As a result of a and b, the soil becomes friable and can be worked readily.

d. Lime aids in drying out wet soils quickly, thus speeding up compaction.

e. The shrinkage and swell characteristics of clay soils are reduced markedly.

f. After curing, UC strength increases considerably--in some instances as much as fortyfold.

g. Load-bearing values, as measured by various tests (CBR, R-value, Texas triaxial, plate bearing or k-value, etc.), increase substantially.

h. The tensile or flexural strength, as measured by various tests (cohesiometer, splitting tensile, etc.) increases markedly. Thus, the stabilized layer develops beam strength.

i. The lime-stabilized layer forms a water-resistant barrier by impeding penetration of gravity water from above and capillary moisture from below. Thus, the layer becomes a firm "working table," shedding rainwater readily and remaining stable, thereby minimizing construction delays.

5.3.3 LIME-FLY ASH AND LIME-CEMENT-FLY ASH STABILIZATION

Fly ash is a by-product of blast furnaces. It is very fine (largely passing a No. 200 mesh screen) residue from burning pulverized coal, and consists of spherical particles composed mostly of silicon and aluminum compounds. Therefore, the addition of fly ash to lime-stabilized soil speeds the pozzolanic action (which is the formation of cementitious silicates and aluminates). Generally, large quantities of fly ash are required for adequate stabilization and thus its use is restricted to areas which have available large quantities of fly ash at relatively low cost.

For nonplastic and low plastic index soils that are unresponsive to lime, a pozzolan (cementitious silicates and aluminates) is needed to produce the necessary lime-silica reaction. Fly ash* is the most commonly used pozzolan for this purpose. Volcanic ash and expanded shale fines have also been used successfully and under certain conditions some reactive clays can be employed.

One of the most important steps in obtaining proper strength gain of lime-fly ash is to allow proper curing (time and temperature) after construction. Barenberg[51] noted that below 40° F the chemical reaction for a lime-fly ash-aggregate mix virtually stops. Above this temperature the rate of reaction increases with increasing temperature. Yang[52] has noted the advantages of the relatively long period of time (5 years) required to achieve ultimate strength gain for lime-fly ash mixes.

Laboratory fatigue tests on lime-fly ash mixtures have been noted in the literature. Barenberg[51] presented a typical fatigue relationship for such a mixture. This response is shown in Figure 5.10. It can be seen that for pozzolanic-type mixes the fatigue relationship can be plotted as a straight line on an arithmetic stress-to-strength ratio versus logarithmic repetition plot.

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* Patents have been issued relating to the use of lime and fly ash in road compositions. Information regarding them may be obtained from Poz-O-Pac Company of America, Plymouth Meeting, Pa.

Yang[53] used a lime-fly ash mixture successfully in stabilizing the hydraulic sandfill in Newark Airport, Port of New York Authority. The details of the construction are explained in the following paragraphs.

The airport site was originally marshy and was surcharged with 14 million cu yd of hydraulic sandfill. Since the fines in the sandfill had been washed away in the process of pumping from barge to reclaimed site, the sand grains were largely of one size (No. 30 to No. 50 mesh). The sand was deficient in the fine particles which are vital in interlocking the granulated soil structure to provide stability under load.

After accurate determination of the properties of all ingredients, the laboratory work led to the decision to use as pavement base a mixture of hydrated lime, portland cement, fly ash, crushed stone (in some of the mixtures), and sand. The fly ash serves, in part, to fill the voids between the large grains of sand. When water is added, the fly ash reacts chemically with hydrated lime, and the mixture hardens as does concrete (though much more slowly).

The mixture had the following composition, by weight:

Hydrated lime	2.8-3.6 percent
Portland cement	0.7-0.9 percent
Fly ash (dry)	12-14 percent
Hydraulic fill sand	About 83 percent (53 percent if stone used)
Crushed stone	30 percent (where used)

Where high-strength (over 2000-psi) base material was required, crushed stone (30 percent by weight) was used in place of some of the sand. The Port Authority named the stabilized material LCF (lime, cement, fly ash).

Since portland cement reacts with soil much faster than lime, the small amount of portland cement in the mixture acts primarily as an additive to accelerate development of chemical bond.

The development of the compressive strength of the LCF mix is directly related to chemical interaction between lime and fly ash, as affected by the temperature of curing. During the spring-to-fall construction season, the growth of compressive strength for the LCF mix with crushed stone was as follows:

	Strength, psi
3 days	Insignificant
1 month	200-400
3 months	600-800
1 year	1000-1200
2 years	1600-2000
Projected	
5 years	2000-2400

For LCF mix without crushed stone, the compressive strength at each time was about two thirds of the tabulated strength.

Because of slow initial setting, there was no need to finish paving work the same day that the LCF material was mixed and spread. For example, material mixed and spread on the second shift was compacted next morning. Man and machine time were conserved. Longer paving seasons and accelerated schedules thus become feasible.

Because of LCF's very slow curing (thus little heat of hydration) and low water content, there was relatively little curing shrinkage and thus little shrinkage cracking.

The advantage of using the lime-fly ash mixture at the Newark Airport was economy. The average cubic yard of LCF material in place at Newark cost \$3.80 (weighted for the percent of LCF with acid without stone used). A cubic yard of crushed stone commonly used in road construction cost \$5.00 to \$6.00. A cubic yard of lean concrete, three-sack mix, cost about \$12.00. If the compressive strength was introduced in judging relative value of a cubic yard of materials:

LCF (average all mixes)	500 psi/\$1.00
Crushed stone	30 psi/\$1.00
Lean concrete	250 psi/\$1.00

The Port Authority believed that LCF was stronger per dollar than any pavement material then in use.

In another paper, Yang[54] reported that the fatigue relationship developed for the LFA mixture used at Newark International Airport was

$$\frac{\sigma_N}{\sigma_b} = 1 - 0.092 \log N \qquad (5.24)$$

where σ_N is the fatigue strength of the material at the N^{th} repetition of the traffic load, σ_b is the static bending (flexural) strength, and N represents the number of repetitions to failure.

Both fatigue responses shown in Figure 5.10 and Equation 5.24 were obtained from direct laboratory tests. However, neither researcher indicated the curing conditions used for the specimens tested in fatigue.

5.4 BITUMINOUS STABILIZATION

Bituminous stabilization is used with noncohesive granular materials where the bitumen adds cohesive strength, and with cohesive materials where the bitumen waterproofs the soil thus reducing loss of strength with increase in moisture content. Both effects stem partly from the formation of films around the soil particles, which stick them together and prevent the absorption of water, and partly from simple blocking of the pores, preventing water from entering the soil mass. To improve penetration and adhesion in the soil, bitumens are usually mixed into the pulverized soil as emulsions, cutbacks, or foams. Bituminous stabilization is generally satisfactory for coarse-grained or granular soils, but its use for stabilizing plastic soils is limited. For instance, an extremely dirty gravel which has considerable quantities of fines and some plasticity may be worsened by stabilization. This results from increasing the plasticity of the material by addition of too much bituminous material.

It should be noted that in the use of bituminous stabilization, there are two opposing effects at work--the thinner the film of bitumen the stronger the material; however, thick films or filled pores are the most effective in preventing ingress of water. Too much bitumen, however, causes loss of strength by lubricating the particles and preventing interlock.

5.4.1 BITUMINOUS MATERIALS

Asphalt and coal tars have been used for bituminous stabilization. They are applied hot or as "cutback" or "emulsion," and recently as "foam." The use of hot bitumen has some obvious disadvantages, and

cutbacks in which the bitumen is in solution in a volatile oil are more frequently used. The temperature of application is lower and, as the volatiles evaporate, the bitumen is deposited on the soil.

Emulsified bitumens are used cold and in slow- or medium-breaking grades. Emulsions consist of a fine suspension of bitumen particles in water, and the bitumen is deposited on the soil when the bitumen suspension coagulates or "breaks."

Foamed bitumens, prepared by blowing steam through hot bitumen to produce a foam of air bubbles in the bitumen, are claimed to assist in compaction and to form uniform films of bitumen on the soil particles.

5.4.2 SOILS

Although heavy clays have been treated with both cutback and emulsified bitumen and have performed adequately, the main use of bitumen stabilization is for sands or sand gravels which lack cohesion and/or where a waterproofing action is required. Any noncohesive soil, sand, and sand-gravel may be treated.

In very clean sands, poor adhesion of the bitumen to the silica surface may permit stripping of the bitumen from the sand if water does penetrate the soil, and the stabilizing effect will then be lost. Wet soils are not generally suitable for processing because of difficulties in mixing and because the addition of more liquid in addition to the water present can render the soil impossible to compact. A particular application to wet sand is possible, however, if the addition of bitumen does not increase the fluid content beyond that necessary for compaction. The "wet-sand" process is applied to fine, nonplastic sands (not more than 3 percent clay) where the addition of 1 to 2 percent hydrated lime promotes adhesion between the bitumen and the sand.

A high concentration of salts or organic matter may reduce the effectiveness of bitumen where the salt or organic matter coats the soil particles and prevents adequate adhesion between the bitumen and the soil.

5.4.3 PROPERTIES OF BITUMINOUS-STABILIZED SOILS

The addition of bitumen to a soil can affect the properties in two ways--by waterproofing the soil and/or by increasing the cohesive strength of the soil. Generally speaking, the strength reaches a maximum value as the bitumen content is raised, but then the strength decreases as the bitumen films become thicker. The higher bitumen content, however, imparts greater waterproofing, and thus it is usual to adopt a compromise between maximum waterproofing and maximum strength.

Addition of the bituminous stabilizer reduces the need for water to be added for compaction, and this may be a considerable advantage in very dry conditions although the maximum density will be reduced. Because, for any compacted density, the higher the bitumen content the less the absorption and loss of strength on soaking compared to the untreated material, and because, beyond a certain level the strength decreases and the material may be impossible to compact when the fluid content has filled the voids completely and thus prevents particle interlock. The total volatile content is important.

Hill and Scott[55] and Gregg, Dehlen, and Rigden[56] investigated bituminous-stabilized sands, using in situ shear tests, in situ CBR tests, laboratory CBR tests, and triaxial tests. The results showed a marked dependence of all these measures on temperature, and a linear relation between temperature and the logarithm of strength was proposed. It was suggested that the triaxial test should be used to assess sand-bitumen mixes and that a value of c (the cohesion intercept) of 30 psi and a value of ϕ (the angle of friction) of 30 deg at a temperature of 25° C would be acceptable. Alternatively, a test at an elevated temperature may be carried out, where a CBR greater than 10 or a vane shear strength greater than 12 psi at 55° C is adequate.

Winterkorn[57] discussed the use of the unconfined compressive strength test using samples compacted to maximum density at the optimum moisture content with a range of bitumen contents. He compared the air-dry strength with that of samples soaked for 7 days and recommended the mix giving the highest dry-wet strength ratio with a minimum strength of 75 psi.

McKesson[58] suggested a method of test for bearing values of sand-bitumen mixtures involving applying a load at a constantly increasing rate 60 psi/min to a 1-in.^{2} pad placed on the surface of a cylinder 4 in. in diameter and 3 in. high. The bearing value is the load in pounds per square inch on the pad when it has penetrated 1/4 in. into the specimen or when radial cracks 3/4 in. long have appeared in the surface, whichever occurs first. The specimens are molded with extra water, which is allowed to evaporate at 60° C before compacting the mix. The method is used for sands finer than the No. 4 sieve and it is recommended that only those sands which have an untreated bearing value in excess of 30 psi should be evaluated in this way.

Alexander and Blott[59] developed the cone stability test to evaluate bituminous-stabilized sands. In particular, this test was used in the investigation of "wet-sand" mixes, showing the rapid gain in stability of such mixes in comparison with dry sand mixes. For the wet-sand process, an arbitrary cohesivity test was also developed by Shell[60] in which the minimum bitumen content necessary to hold together a pat of sand in water as the water is stirred is determined. This proportion of binder is, however, not considered adequate in practice and bitumen contents are increased by 1-1/2 percent. The elasticity of bituminous-stabilized mixes is of the same order as conventional base materials and therefore no reduction in pavement thickness is possible.

Effect of temperature on the stability of bituminous-stabilized soil was studied in India.[61] The results showed that an increase in temperature at the time of mixing and compacting soil-cutback mixes increased the stability and decreased the water absorption and expansion in the modified Hubbard-Field test. The rate of increase of stability decreased above 40° C. At a particular temperature, an optimum mixing period may be defined also.

Bowering[62] reported the results of the use of foamed bitumen for soil stabilization and the increase in stability after a loss of volatiles. He found a threefold increase in CBR for a nonplastic dune sand stabilized with 6 percent foamed bitumen and compacted immediately.

The test procedures suggested therefore include a curing period after compaction to allow for evaporation. The soil-bitumen mixtures tested show the usual pattern of behavior and Bowering proposes the use of a "resistance value" test and a "relative stability" test, which measure the resistance value at 140° F, in addition to the use of the Hveem cohesiometer and the usual UC strength and CBR tests. The various stability tests all show maximum values which are reduced by soaking or by exposure to water vapor at a temperature of 140° F. His permeability and swell tests show decreasing values with increasing bitumen content as shown earlier for absorption and expansion tests.

5.5 TESTING EQUIPMENT AND TECHNIQUES

Various types of tests have been used to evaluate the properties of stabilized soil mixtures. These methods include the following:

- a. UC strength, static and repetitive.
- b. Flexural strength, static and repetitive.
- c. Indirect tensile tests.
- d. Direct tensile tests.
- e. Creep tests.
- f. Freezing and thawing tests.
- g. Shrinkage tests.

Details of test methods and equipment can be found in many references[1,4,5,23-29,32,63-65] and are not presented in this report.

Table 5.1

Admixture Stabilization (after Yoder[1])

	Type	Admix	Primary Mechanics of Stabilization	Use	Situations Best Suited	Approximate Quantity by Weight	Method of Evaluation	Change Soil Property: γ	LL	PL	PI	Construction Procedure	Limitations
Increase Strength by Cementing Action	Cementing agents	Portland cement	Principally hydration. Some modification of clay minerals	Base and subbase	Sandy soils or lean clays	A-7 9–15% to A-2 5–9%	Durability tests, compression	Decrease*	Slight reduction	Increase	Decrease	Pulverize, mix compact, cure	Organic soils
		Lime	Change water film, flocculation, chemical	Some base and subbase, shoulders	Granular materials or lean clays	2–5%	Do	Decrease*	Varies	Increase	Decrease	Do	Freeze-thaw may be destructive
		Lime flyash	Pozzolanic action of lime and silica, some modification of clay minerals	Some base and subbase, shoulders	Granular materials or lean clays	2–5% lime 10–20% flyash	Do	Decrease*	Varies	Increase	Decrease	Do	Quantity of flyash may be high
		Sodium silicate	Solidification by gel	Deep foundations	Sands		Special laboratory tests					Injection	Rate of set
Improve Plasticity May or May Not Increase Strength	Modifiers	Cement	Modification of clay, Change water film	Improve poorly graded base and subbase	Improve existing road metal, clays	½–4%	Atterburg limits Grain size		Varies	Increase	Decrease	Pulverize, mix, compact	Strength increase may be low
		Lime	Change water film, modification of clay minerals	Do	Do	½–4%	Do		Varies	Increase	Decrease	Do	Do
		Bitumen	Retards moisture absorption	Do	Improve existing road metal	1–3%	Absorption tests					Do	Do
Little or No Increased Strength	Water-proofing agents	Bitumen	Retards moisture sorption by coating soil grains	Primarily subbase	Sandy soils or poor quality base materials, some clays	4–6%	Water sorption, compression, volume change	Increase				Mix, "cure," compact	Limited by soil plasticity
		Membranes	Prevents movement of free water and water vapor	Primarily subbase	Soils that may be improved by compaction		Strength of natural soil	None	None	None	None	Compact to high density	Construction may be difficult
	Water-retaining agents	Calcium chloride	Deliquescent properties, lowers freezing point, base exchange	Construction expedient, traffic binding	Graded aggregate	½–1½%	Arbitrary	Slight increase	Slight reduction	None		Spread dry or mix with water	Leaches
		Sodium chloride	Deliquescent properties, lowers freezing point	Do	Do	½–1½%	Arbitrary	Do	Do	Do		Do	Leaches
Little or No Increased Strength	Water retarding	Organic cationic compounds	Alters clay minerals to act as a hydrophobic agent	Subbases		Trace	Water sorption, compression, volume change					Mix and compact	Mixing very small quantities may be difficult
	Misc. chemicals	Resin Calcium acrylate Sulphite lignin											Limited to special applications

* In some cases a slight increase is shown for granular soils.

Table 5.2

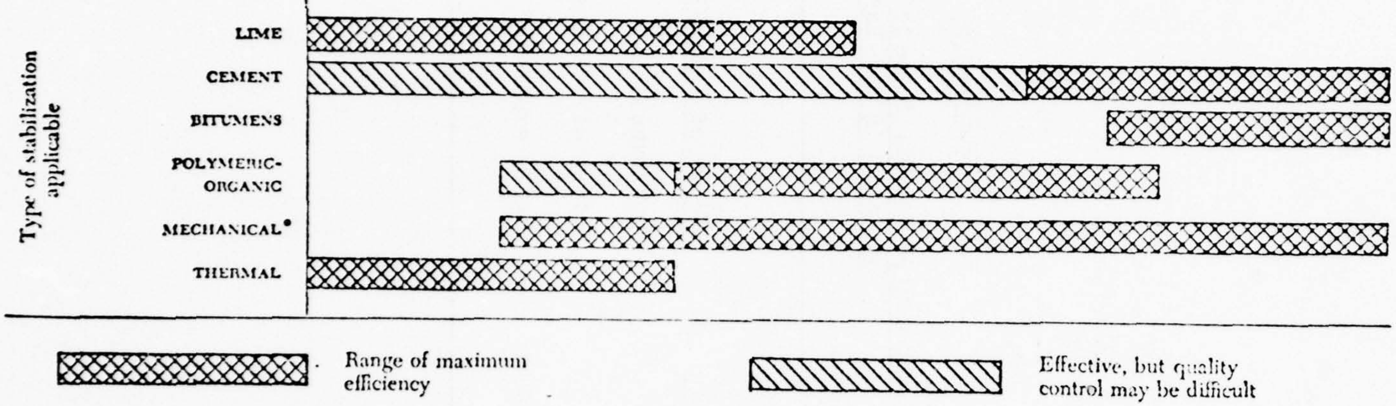
Relationships Between Plasticity and Expansion of Soils (after Packard[2])

Plasticity Index (ASTM D 424)	Degree of Expansion	Approximate Percentage of Swell (ASTM D 1883)
0 to 10	Nonexpansive	2 or less
10 to 20	Moderately expansive	2 to 4
More than 20	Highly expansive	More than 4

Table 5.3

Applicability of Stabilization Methods (after Ingles and Metcalf[3])

Designation	Fine clays	Coarse clays	Fine silts	Coarse silts	Fine sands	Coarse sands
SOIL Particle size (mm)	<.0006	.0006–.002	.002–.01	.01–.06	.06–.4	.4–2.0
SOIL Volume stability	V. poor	Fair	Fair	Good	V. good	V. good



* i.e. improvement of soil grading by mixing-in gravels, sands or clays as appropriate

Table 5.4

Cement Requirements of AASHTO Soil Groups for Soil-Cement

AASHTO Soil Group	Usual Range in Cement Requirements	
	Percent by Volume	Percent by Weight
A-1-a	5- 7	3- 5
A-2-b	7- 9	5- 8
A-2	7-10	5- 9
A-3	8-12	7-11
A-4	8-12	7-12
A-5	8-12	8-13
A-6	10-14	9-15
A-7	10-14	10-16

Table 5.5

Range of Unconfined Compressive Strengths of Soil-Cement (after Reference 4)

Soil Type	Wet Compressive Strength, psi* 7 days	28 days
Sandy and gravelly soils:		
AASHTO Groups A-1, A-2, A-3 Unified Groups GW, GC, GP GF SWISC, SP, SF	300-600	400-1000
Silty soils:		
AASHTO Groups A-4 and A-5 Unified Groups MH and CL	250-500	300- 900
Clayey soils:		
AASHTO Groups A-6 and A-7 Unified Groups MH and CH	200-400	250- 600

* Specimens moist-cured 7 or 28 days, then saturated in water prior to strength testing.

Table 5.6

<u>Typical Properties of Cement-Stabilized Soils (after Ingles and Metcalf[3])</u>

Soil Type	Strength Range [1] kgf/cm^2 (lbf/in^2)	'E' Value [2] kgf/cm^2 (lbf/in^2)	C.B.R. [3]	Permeability cm/sec.	Thermal Expansion [4] in./in./°C	Volume Change [5]	Comments	Use
Well graded gravel-sand-clay, sands or gravels	28-105 (400-1500) and more. Ratio of wet/dry strength 1:1.5	$7\text{-}21 \times 10^4$ ($1\text{-}3 \times 10^6$)	More than 600	High. Decreased by cement 15×10^{-3} unstabilized 18×10^{-6} stabilized		Very small less than 1% (Concrete 0.1%)	Too strong. Wide-spaced wide cracks. Suitable for bituminous stabilization	Base for heavy traffic
Silty sands; sandy clays; sand and gravel	17-35 (250-500)	7×10^4 (1×10^6)	600	High. Decreased by cement		Small	Good material.	Base for heavy traffic
Silty-sandy clays, poorly graded sands	7-17 (100-250)	$3.5\text{-}7 \times 10^4$ (5×10^5 – 1×10^6)	200	5×10^{-6} unstabilized 0.1×10^{-6} stabilized	10×10^{-5} 7×10^{-5}		Compaction difficulties in sands	Sub-base for light traffic
Silts, silty clays, very poorly graded soils	3.5-10.5 (50-150)	Less than 3.5×10^4 (Less than 5×10^5)	Up to 100	Low. Increased by cement		Moderate		Low-grade sub-base
Heavy clays; organic and sulphate rich soils	< 7 (< 100) Ratio of wet/dry strength 1:3	Up to 1.4×10^4 (Up to 2×10^5)	Up to 50	Very low (10^{-11}) Increased by cement (10^{-9})	10×10^{-6}	High > 4%. May be increased by cement	Extreme difficulty in mixing. Use of lime could be beneficial. Special treatment for organic and sulphate soils.	possibly for upgrading subgrades

NOTES:

[1] Strength given as approximate figure for 7 days cure at constant temperature and moisture content appropriate economic cement content, density and moisture content levels. Ratio of U.C.S. to flexure about 4:1 in sand; 3:1 in kaolin. Ratio of U.C.S. to I.T.S. about 10:1.

[2] From flexure tests. Ratio of static to dynamic values about 1:1. Poissons ratio ranges from 0.1 to 0.3.

[3] Approximate figures for mixes with 7 days U.C.S. of 250 psi (accepted U.C.S. for base construction).

[4] Coefficient for concrete 3 to 8 x 10^{-6}; bitumen 6 x 10^{-4}.

[5] Only very limited data.

Table 5.7

Illustrative Values of the Elastic and Strength Properties of Soil-Cement Mixtures (after Felt and Abrams[18])

Soil	Cement Content, percent		Values at 28 days (Moist-Cure), 10^6 psi			
			Compressive	Modulus	Modulus of Elasticity	
	By Weight	By Volume	Strength	Rupture	Dynamic	Static
Sand	3.8	5	450	110	2.05	
	6.0	8	800	180	2.75	
	8.5	11	1225	260	3.30	
Sandy loam	3.8	5	300	80	1.40	0.90
	6.1	8	650	145	2.00	1.25
	8.6	11	1025	215	2.60	1.65
Clayey sand	5.7	7	475	105	1.30	
	8.3	10	625	150	1.50	
	11.0	13	800	195	1.75	
Silt loam	8.0	9	525	125	0.90	0.55
	11.1	12	725	155	1.05	0.65
	14.2	15	900	190	1.25	0.75

Note: The lowest cement content listed for each soil is the quantity required to produce soil-cement that will meet wet-dry, freeze-thaw criteria for base course construction.

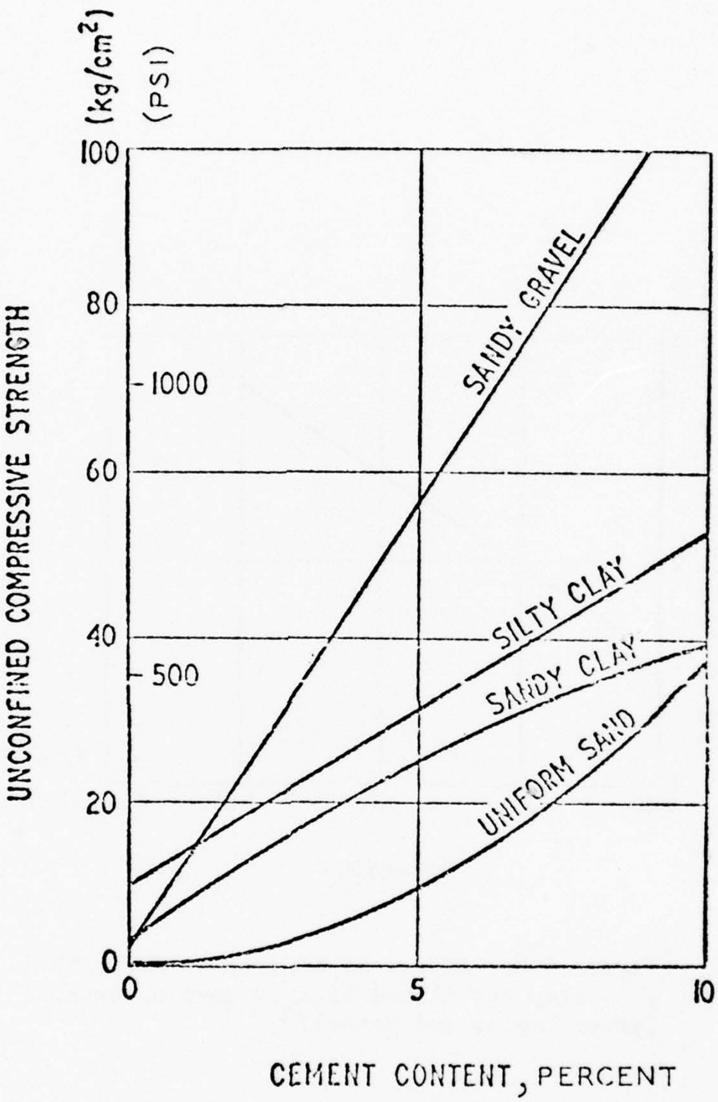


Figure 5.1. Effect of cement content on strength for various soils stabilized with ordinary portland cement, and cured for 7 days at 25° C, constant moisture content (after Metcalf[11])

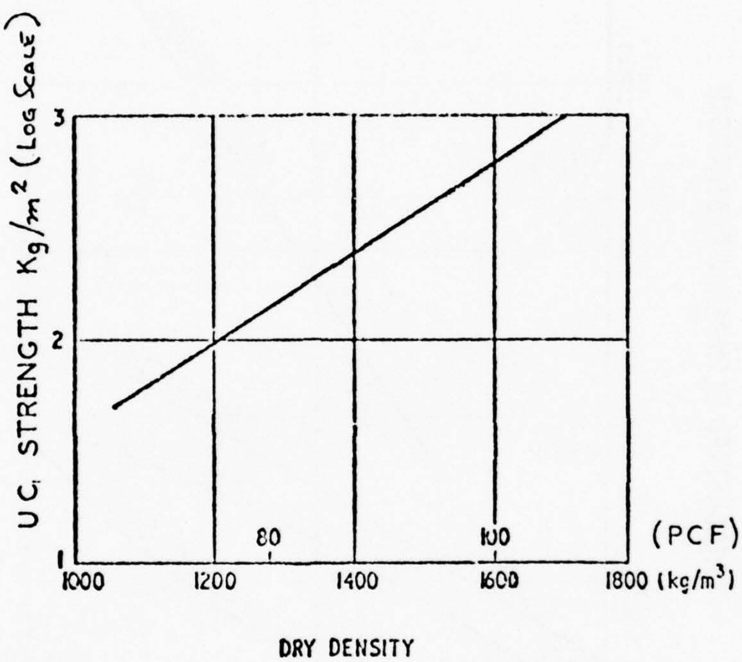


Figure 5.2. Effect of density on strength of a clay stabilized with 10 percent cement (after Ingles and Metcalf[3])

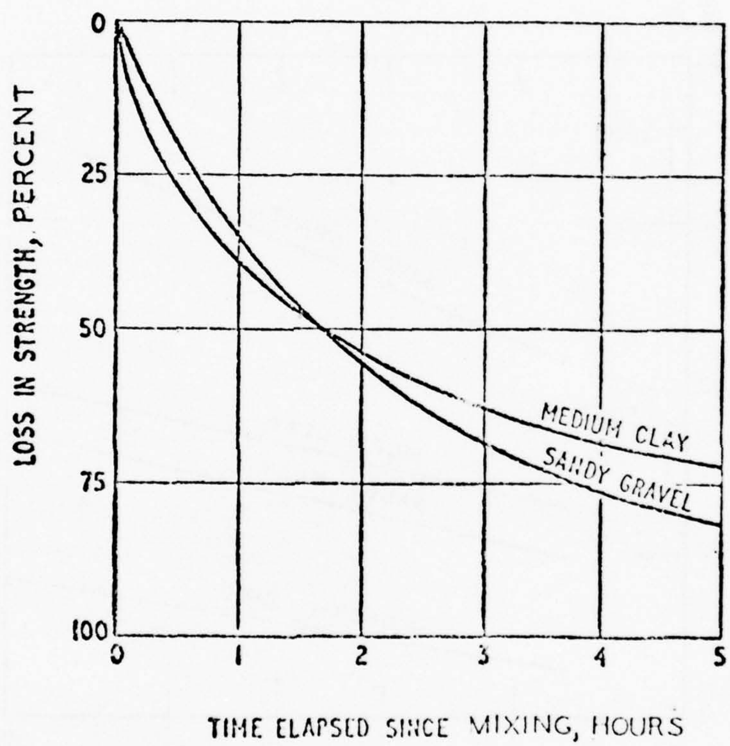


Figure 5.3. Loss in strength due to delay in compaction for two soils stabilized with 10 percent cement; standard compaction (after West[10])

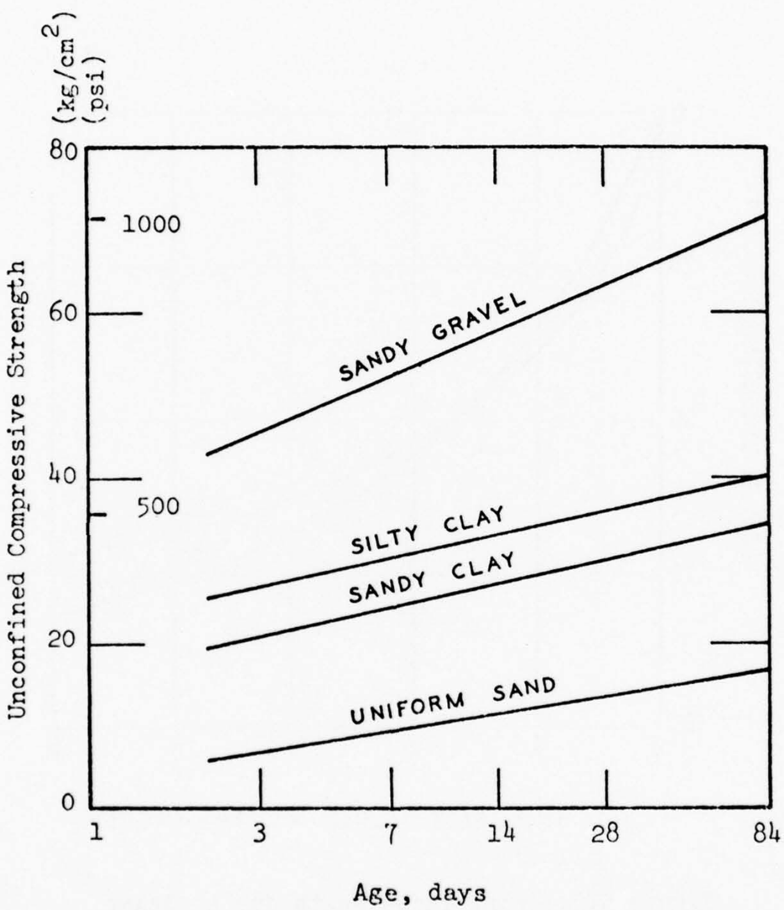


Figure 5.4. Effect of age on strength of various soils stabilized with 5 percent cement (after Metcalf[11])

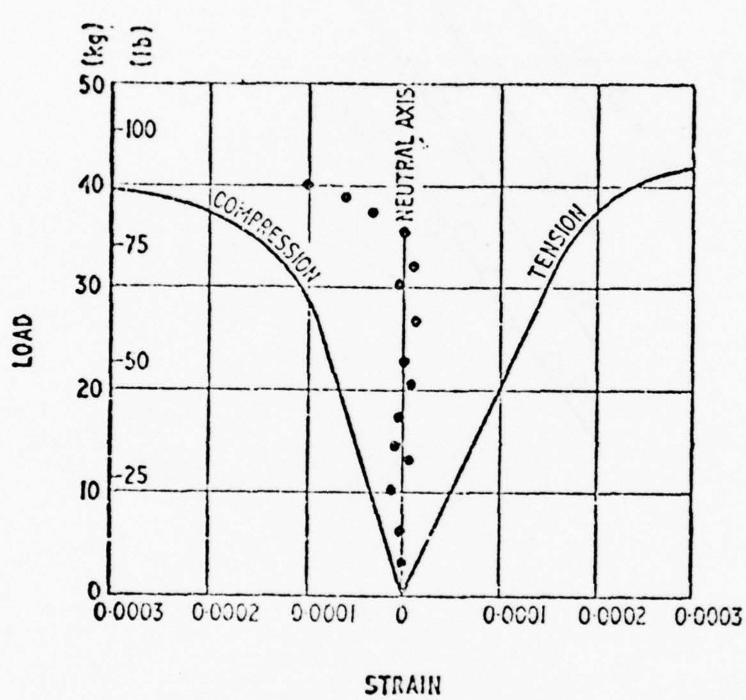


Figure 5.5. Flexural stress-strain behavior of a clay stabilized with 10 percent cement showing linear behavior to two thirds of failure load (after Metcalf[20])

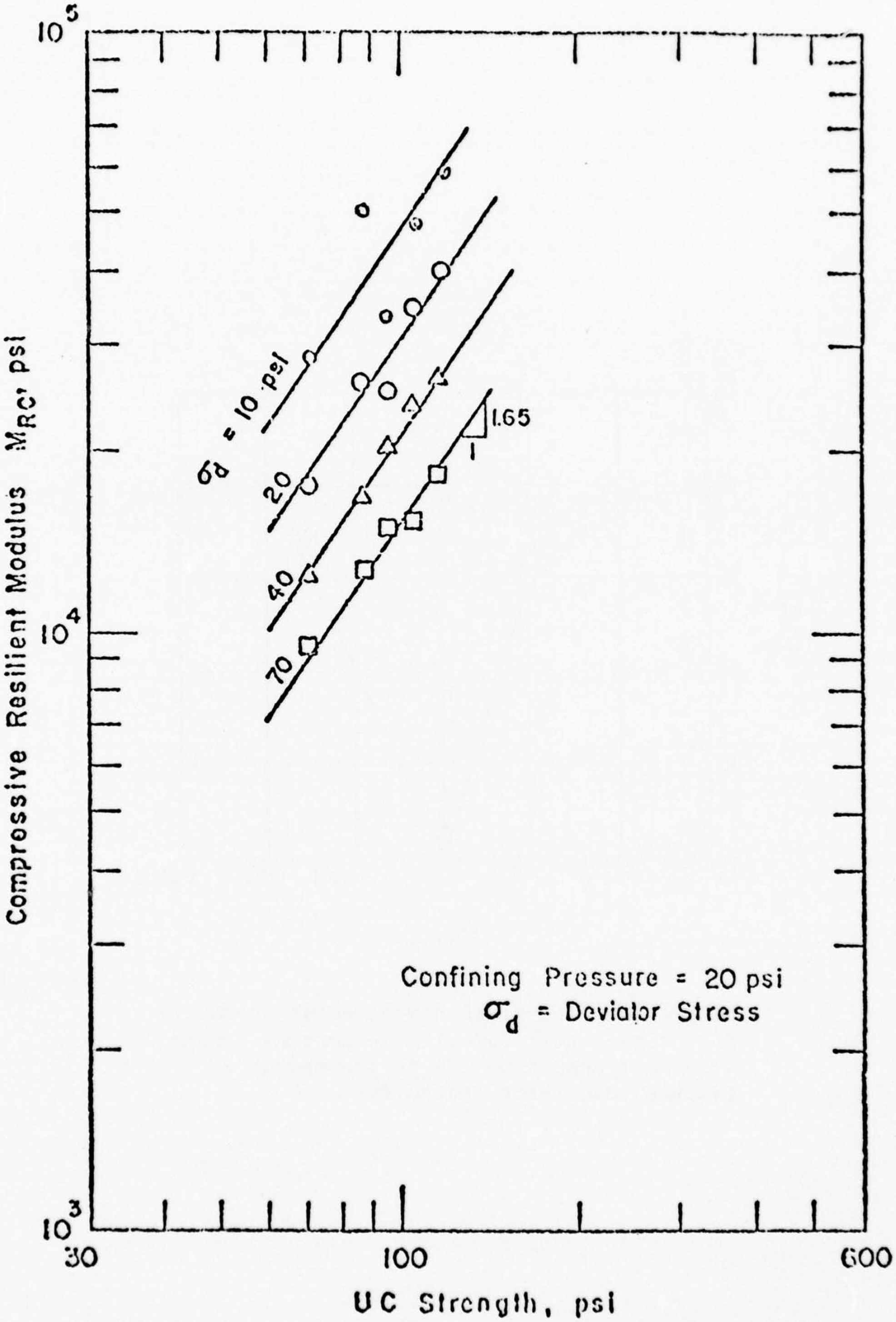


Figure 5.6. Relation between compressive resilient modulus and UC strength for Vicksburg silty clay treated with 3 percent cement (after Mitchell, Jeng, and Monismith[13])

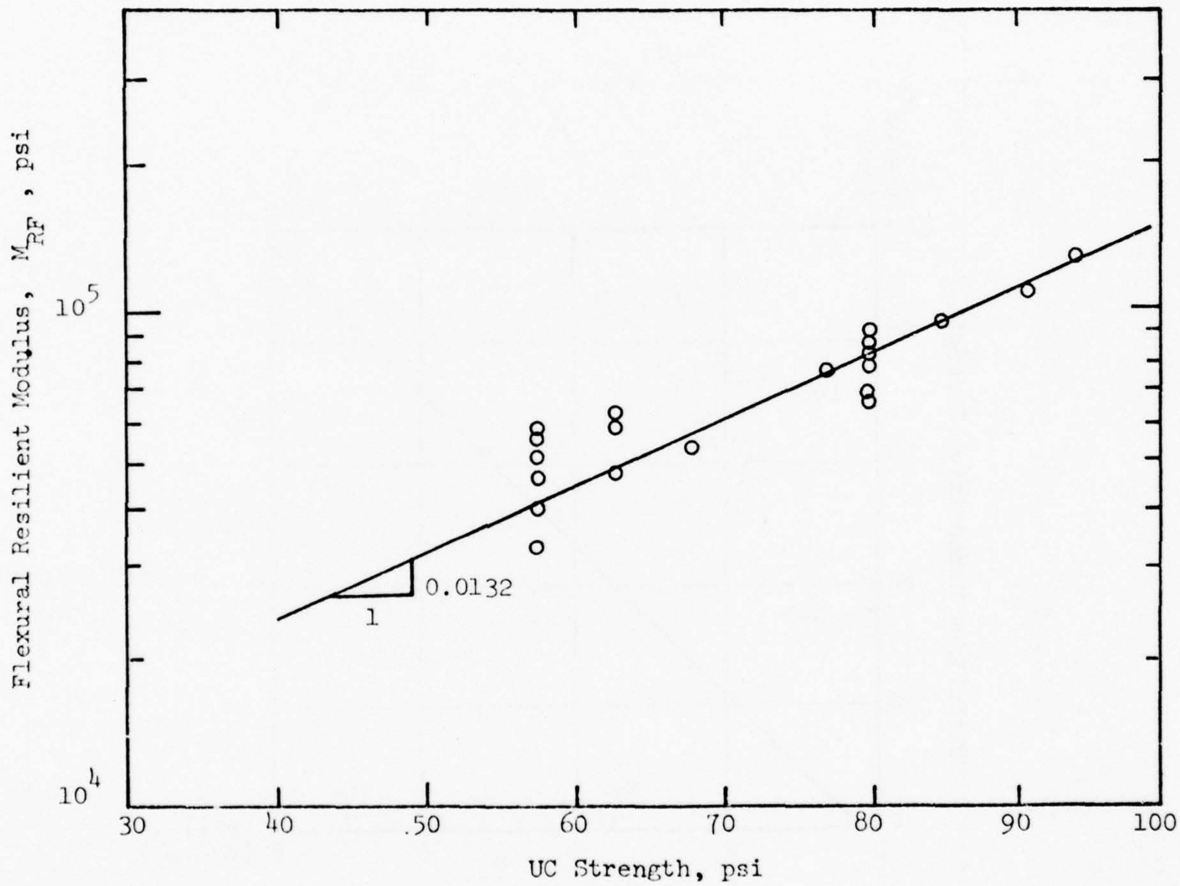


Figure 5.7. Relationship between flexural resilient modulus and UC strength for Vicksburg silty clay treated with 3 percent cement (after Mitchell, Jeng, and Monismith[13])

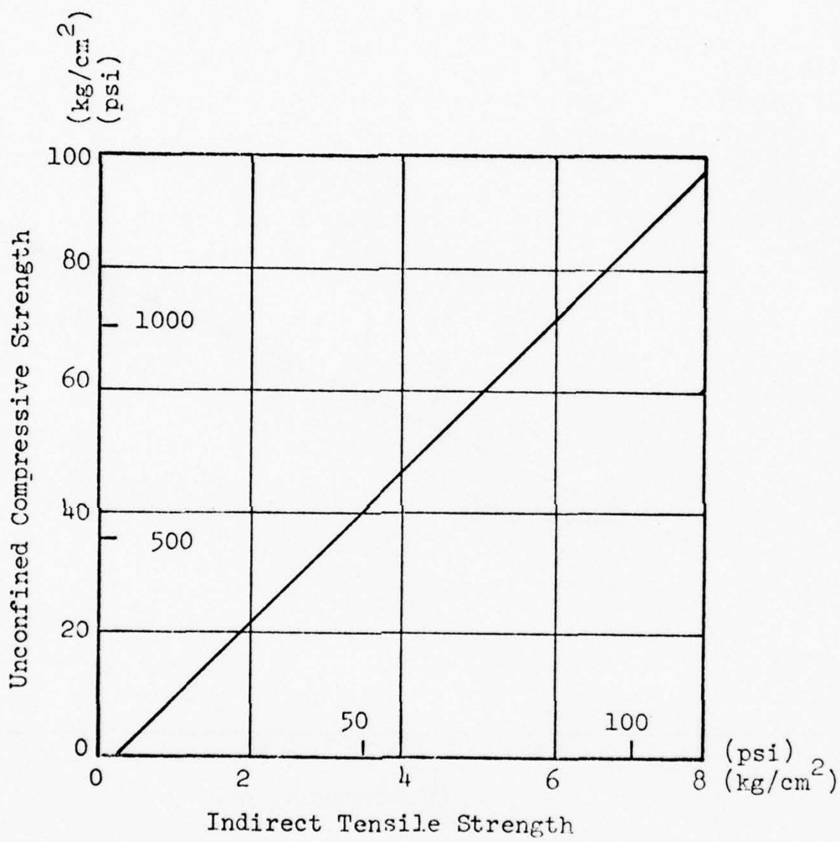


Figure 5.8. Relation between compressive and tensile strength for cement-stabilized soils (after Metcalf[32])

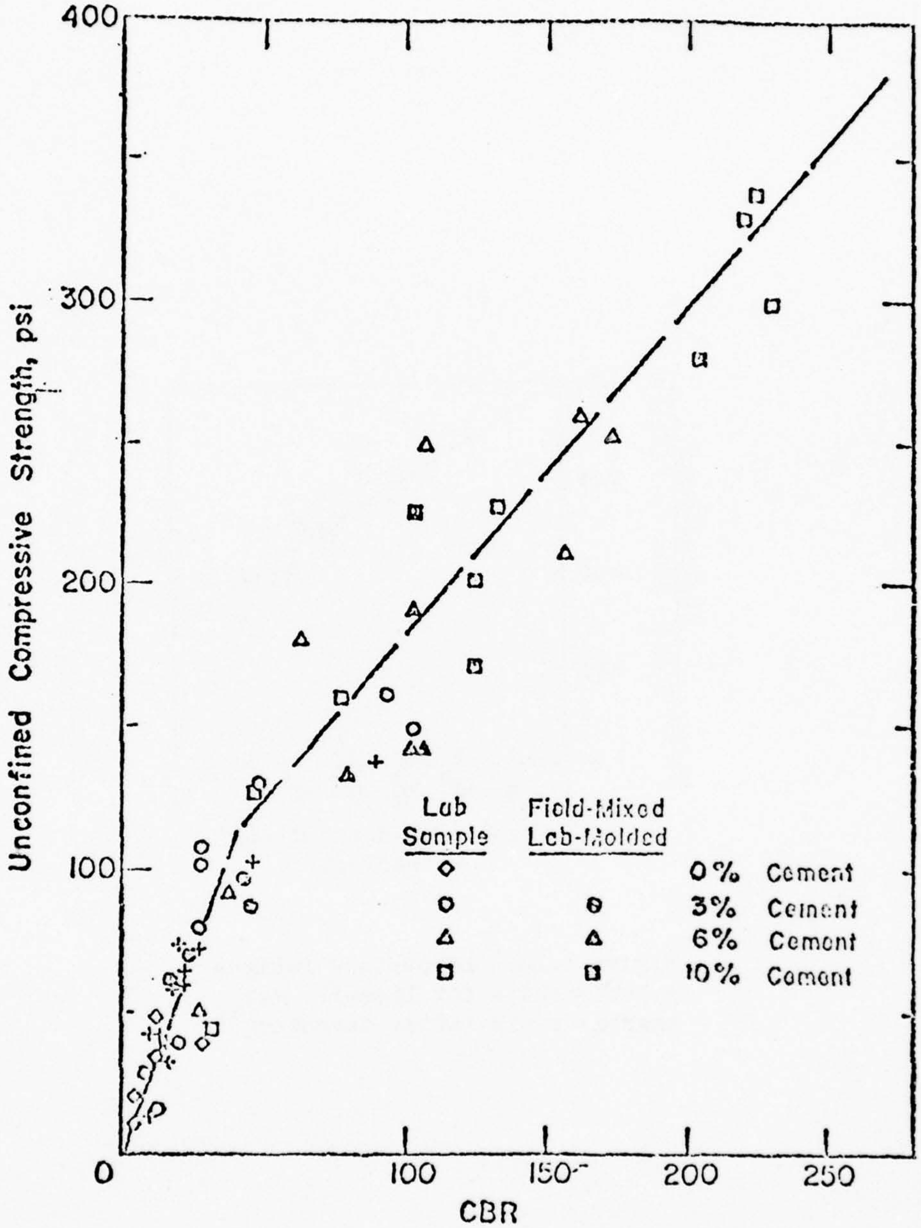


Figure 5.9. Relationship between CBR and UC strength for cement-stabilized Vicksburg silty clay (after Mitchell, Jeng, and Monismith[13])

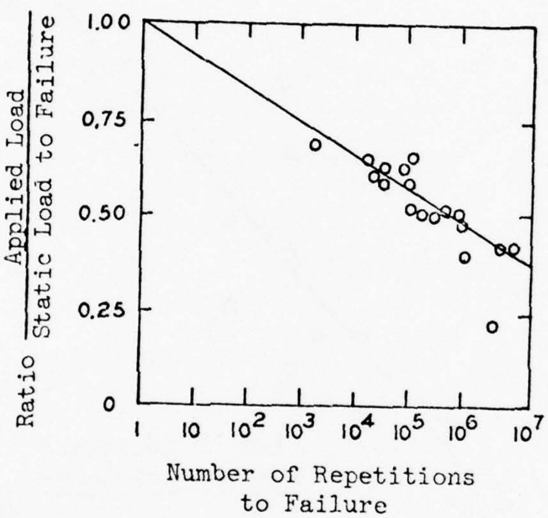


Figure 5.10. Laboratory fatigue relationships for lime-fly ash-aggregate mix (after Barenberg[51])

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NOTATION

a	Factor varying with thickness of pavement h
A	Constant
b	Constant
c	Cohesion
C	Cement content, percent by weight
D	Density
E	Modulus of elasticity
E_r	Resilient modulus
$g(\sigma_d)$	Function of deviator stress
h	Thickness of pavement
I_1	First stress invariant, psi
K	Constant dependent on type of soil and cement content
K_c, K_F	Constant dependent on material
K_1, K_2	Exponents
LL	Liquid limit
m	Exponent
M_{Rc}	Resilient modulus in compression
M_{Rf}	Resilient modulus in flexure
n	Exponent $= 1.0 + 0.18C$
N	Number of load repetitions
N_f	Number of load repetitions at failure
PI	Plasticity index
PL	Plastic limit
R	Allowable radius of curvature for given number of load repetitions
R_c	Critical radius of curvature; i.e., the minimum curvature at failure under static load
S	Strength; also, stress level
UC	Unconfined compressive strength, psi
$(UC)_D$	Unconfined compressive strength after D days curing
$(UC)_{D_o}$	Unconfined compressive strength after D_o days curing

γ	Density
ε_i	Initial flexural strain based on strain gage measurements
ν	Poisson's ratio
σ_b	Static bending (flexural) strength
σ_d	Deviator stress
σ_i	Initial flexural stress
σ_N	Fatigue strength at N^{th} repetition of traffic load
σ_1	Major principal stress, psi
σ_3	Confining pressure, psi
ϕ	Angle of internal friction

APPENDIX A: U. S. AIR FORCE SOIL STABILIZATION INDEX SYSTEM

In early 1970, a research program was initiated by the Air Force Weapons Laboratory to develop a soil stabilization index system (SSIS) to aid military engineers in selecting the appropriate type and amount of soil stabilizer to use in pavement construction. The research was conducted at Texas A&M University. The system was to be arranged in such a form and with sufficient background information that it could be effectively used even by engineers not specifically trained in stabilization techniques. Insofar as possible, SSIS was also to consider factors influencing soil stabilization other than soil properties, such as the urgency of construction, the location of the stabilized layer in the pavement, the type of construction equipment available or needed, and the influence of environmental conditions on the stabilized layer.

Reference 66 contains the index system and the basis for its development. The index system is entered with easily determined soil properties, and flow charts are used to determine specific amounts of stabilizers (only lime, portland cement, and bituminous stabilizers were considered). Use factors, construction factors, and environmental factors are also considered in the decision-making process. Since the index system was based on a comprehensive review of published information and personal opinions of acknowledged experts in the soil stabilization field, conflicting viewpoints existed in many places, necessitating validation of the proposed system. Reference 67 covers the validation of the SSIS based on laboratory tests and discussions with experts in soil stabilization. Based on these tests and discussions, several changes were made to the initial index system, while the original concept was not altered.

CHAPTER 6: COHESIVE SUBGRADE SOILS

6.1 INTRODUCTION

A pavement is designed to provide a smooth and strong surface over which vehicles may pass under all climatic conditions for a designed period of time. In turn, the performance of the pavement is affected by the characteristics of the subgrade. Desirable properties which the subgrade should possess include strength, drainage, ease of compaction, permanency of compaction, and permanency of strength. It has been shown that soil strength is a function of soil type, moisture content, and density, and these factors are interrelated. The design of subgrades involves a thorough study of the strengths of soil under both static and repeated load conditions, and the establishment of density and moisture-content requirements to be specified for construction.

Certain types of soils are undesirable to be used as subgrade soils, and should either be totally removed or be given specific considerations if they have to be used. These are:

a. Soils which contain large quantities of mica or organic materials are elastic and subject to rebound upon removal of load.

b. Soft and organic soils.

c. Frost-susceptible soils in cold climates.

d. Soils which exhibit properties of high volume expansion upon wetting and shrinkage upon drying.

6.2 RESILIENT PROPERTIES

Extensive studies of the behavior of fine-grained materials in laboratory repeated-load tests have been made by Seed and his associates.[1-7] In these investigations, soils were subjected to repeated loads of durations corresponding to those which occur in actual pavements and to frequencies of load application from 20 per minute to approximately 3 per hour.

Typical results are shown in Figure 6.1, together with a plot of the resilient modulus as a function of the number of load repetitions.

From investigations covering a range in conditions and soil types, the factors influencing the resilience of clays under repeated loads are summarized below. The materials are taken from References 8 and 9.

a. Number of stress applications. As may be seen in Figure 6.1, the resilient deformations generally decrease as the number of load repetitions increases. Thus, deformations determined under a relatively small number of stress repetitions may present a misleading picture of the resilience characteristics of the subgrade soil.

b. Age at initial loading (thixotropy). Samples compacted to high degrees of saturation increase in strength with time. The resilient strain determined for small numbers of stress repetitions decreases as the time interval between compaction and testing increases. However, after large numbers of repetitions, because of thixotropic changes and deformations occurring during repeated loading, the effects of aging are reduced and essentially the same results are obtained for specimens tested immediately after compaction as for specimens tested after a delay.

c. Stress intensity. When analyzing stresses and deflections in pavements, the influence of the intensity of stress in repeated loading is particularly important. As shown in Figure 6.2, at low stress levels, the resilient modulus decreases rapidly with increasing values of the deviator stress, with a variation of over 400 percent as the stress increases from 3 to 15 psi. However, at stresses above 15 psi, there is only a slight increase in resilient modulus with further increases in deviator stress. Unfortunately, from a design point of view, the stress levels to which pavement subgrades are subjected are likely to be in the lower ranges where the resilient modulus varies widely. As the depth of a soil element below the pavement surfaces increases, the deviator stress will progressively decrease and thus, even if the soil were completely uniform, the resilient modulus would in fact increase with depth. This variation will clearly complicate the application of elastic theories developed for conditions of uniform moduli for the computation of resilient pavement deflections, and will require careful consideration in the selection of a single modulus value for incorporation in such theories. It also indicates that the contribution of the upper layers of a compacted clay subgrade to the total resilient subgrade deflection will be far greater than would be indicated by elastic theory.

d. Method of compaction. Methods of compaction which tend to produce dispersed structures in soils tend to produce lower moduli of resilient deformation.

e. Compaction density and water content. The compaction conditions have influence on the resilience characteristics of fine-grained material. As the degree of saturation at compaction increases, the resilient deformation at a particular stress level increases and the resilient modulus decreases.

f. Changes in water content and density after compaction. In general, as the water content of the soil increases due to water absorption after placement, the resilience increases; on the other hand, as the density increases, the resilience decreases.

Another approach to the determination of the elastic properties of subgrade soils has been described by Coffman, Kraft, and Tamayo.[10] Specimens of the AASHTO Test Road subgrade soil were subjected to creep tests in simple axial compression under a range in stresses from 9.0 to 18.0 psi and the relationship between deformation and time was determined. The creep data were transformed by numerical methods to obtain the complex modulus* for the material. Typical values for the modulus as a function of frequency are shown in Figures 6.3 and 6.4. A brief comparison between the data obtained by Coffman, Kraft, and Tamayo[10] and Seed, Chan, and Lee[8] is given in Table 6.1. Although some discrepancies are noted, the order of magnitude of the results for one soil is essentially the same. The samples prepared by Coffman, Kraft, and Tamayo were also compacted by kneading compaction. In their report, it was noted that the dwell time of the tamping foot varied from 1.6 to 5.5 sec, which was longer than that for the compactor used by Seed, Chan, and Lee. It is thus possible that more dispersion was introduced in these samples because of this longer dwell time and, as noted earlier, with increased dispersion a higher deformation, and accordingly lower modulus, are obtained. This, together with the lower confining pressure acting on the test specimens, could account for much of the difference noted.

Hveem et al.[11] performed another type of transient test to obtain an indication of the resilient behavior of soils. Using a modified stabilometer, the deformation of a standard sample in repeated loading

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* The complex modulus is explained in Chapter 2.

was measured as volumetric displacement. Using specimens prepared according to the California method of compaction, the influence of compaction water content was determined for a variety of materials. Hveem's results indicate the same trends as those obtained in Seed's investigations; i.e., an increase in resilience with increased molding water content. Hveem has also demonstrated the influence of thixotropy and the influence of deviator stress.

In References 12-14, Robnett and Thompson report the results of resilient triaxial tests for samples of a number of different soil types. Results indicate that the resilient properties of fine-grained soils range over a wide spectrum. It was found that such parameters as degree of saturation and volumetric moisture content account for a substantial portion of the variability in resilient properties. Based on the data from these tests regression equations were developed (Table 6.2) for predicting resilient modulus for conditions of 95 percent AASHTO T-99 compaction and optimum moisture content. To correct the resilient modulus for different moisture content conditions (compaction still at 95 percent), the resilient modulus is adjusted 0.334 ksi for each 1 percent change in percent saturation. Other regression equations are presented by which the resilient modulus can be predicted from the degree of saturation, the volumetric water content, or the soil classification. Strength measurements (CBR and unconfined compression tests) were also conducted in conjunction with the resilient modulus test. As to the relationship of resilient modulus to CBR, Reference 12 presents the following conclusions:

> Based on the data, analyses, and discussions presented in this report it is apparent that Illinois soils display a wide range of resilient characteristics and that the CBR procedure is not adequate for evaluating the subgrade support of fine-grained Illinois soils subjected to repeated, rapidly applied loads of short duration.

In addition, it is reported in Reference 13 that a strong positive correlation was found for the static stress-strain data (unconfined compressive strength and static modulus).

The conclusion in regard to the correlation of resilient modulus and CBR is particularly significant in view of the use of the relationship of 1500 × CBR which has been suggested in several design procedures (References 15 and 16). In view of the wide usage of the relationship between CBR and resilient modulus, the correlation study presented in References 12-14 should be examined in more detail. In Reference 13 it is noted that the soaked CBR is the strength value which correlated negatively with resilient modulus whereas the immediate CBR had a positive correlation with resilient modulus. Concern is felt by this reviewer that in the presentation of the data (Reference 14) the moisture contents are not given for the CBR values, either the soaked CBR or the immediate CBR, nor is it indicated in the reports that these moisture content determinations were even made. In view of the sensitivity of both the resilient modulus and the CBR to moisture content, it would seem that comparisons between resilient modulus and CBR must be made such that the effect of moisture content is taken into account. Considering the moisture-density-CBR relationships for a lean clay (CH) as shown in Figure 6.5, it is seen that as the molding water content is increased there is a drastic drop in the CBR (unsoaked CBR). The drop in CBR with increases in molding water content is more severe for the greater compactive effort; in fact, above a molding water content of 15 percent the lower compactive efforts have the higher CBR's. This behavior is indicative of a material which is sensitive to repeated shear stresses. Next consider the plot of the soaked CBR versus water content for the same soil (Figure 6.6). From this plot it is seen that up to about the optimum water content the soaked CBR actually increases with increases in molding water content. Past the optimum there is a severe decrease in the soaked CBR which is reflecting the sensitivity to shear stress at the higher molding water content.

The behavior of a CH (Figures 6.7 and 6.8) material is different from the behavior of the CL in that it does not exhibit the external sensitivity to molding water content and shear stress as did the lean clay. It is noted that there is a gradual decrease in CBR (unsoaked CBR)

with increase in molding water content and that above the optimum molding water content the CBR is about the same for all compactive efforts. The soaked CBR shows a reverse trend from the unsoaked CBR; i.e., for increases in molding water content there is an increase in the soaked CBR. Thus it can be seen that for this material a negative correlation would be expected between resilient modulus which decreases with molding water content, and a soaked CBR which increases with molding water content.

WES conducted resilient modulus tests on soils from a number of different locations at which field CBR tests had been conducted. The field data included the field CBR, water content, and in-place density. In the laboratory, gradation and limits determinations were made and resilient triaxial tests were conducted. The resilient triaxial tests were conducted on undisturbed samples for the locations for which such samples could be obtained. Otherwise, samples were remolded to the approximate density and water content of the in-place material.

The resilient triaxial tests were conducted at confining stresses of 2.5, 5.0, and 10.0 psi and over a range of deviator stresses. Examples of the test data for a site are shown in Figures 6.9 and 6.10. In the design procedure presented in Reference 15 the design modulus is obtained by using the data given in Figure C5 of Reference 15 and the resilient modulus data from the laboratory tests. Table 6.3 provides a comparison of the design modulus (6000 annual departures) determined by the above method and the modulus obtained by the relationship of 1500 × CBR. From the table it is seen that favorable comparison is obtained for the low CBR's (less than 6.8) for which the design stress is low but a not-so-favorable comparison for the high CBR's (58 percent error for the 20 CBR MESL) for which the design stress is high. Even though an error of 58 percent seems large, Reference 15 points out that for one particular soil a 50 percent reduction in resilient modulus can be obtained by increasing the moisture content from 23.6 to 27.4 percent.

The data presented by WES, Thompson and Robnett, and other researchers indicated that the resilient modulus and the CBR of most

soils have a high degree of sensitivity to changes in moisture content. Also, some silty type soils indicate, at moisture contents above the optimum, a sensitivity to repeated shear stresses. Thus it appears that selection of the test conditions, i.e., soil moisture contents, test procedures, and test equipment, can dictate the relationship between resilient modulus and CBR. Where Thompson and Robnett found negative correlation between soaked CBR and resilient modulus of soils near optimum water content, WES found a good correlation between field CBR and resilient modulus, provided both tests are conducted at the same moisture content.

6.3 PLASTIC PROPERTIES

Recently, Monismith, Ogawa, and Freeme[19] conducted a series of repeated load tests on fine-grained soils to ascertain the effects of compaction conditions, stress magnitude, and stress sequence on the accumulation of permanent strain with repeated stress repetitions.

Test results were plotted semilogarithmically with axial, radial, and volumetric strain against the number of stress repetitions. Plots of change of strain per cycle showed that the rate of strain decreases with increasing load repetitions, and that permanent strain increased with increasing deviator stress. The results also showed that specimens compacted near the maximum dry density tended to deform less.

The tests to investigate stress history showed that specimens subjected to small levels of stress before being subjected to greater stress levels deformed less than those without the conditioning stress. A test series in which various combinations of 3, 5, and 10 psi were applied to specimens, showed again that when the smaller stresses are applied first the specimen deforms less.

When results were plotted on a log-log basis, straight lines were obtained, as in Figure 6.11. These log-log plots strongly resemble similar ones plotted for bituminous material as shown in Figure 2.21 of Chapter 2, and the form of the equation developed to represent them is the same.

In practice, subgrades are subjected to lower stress levels than those to which the specimens were subjected, since these were chosen

so that measurable strains could be obtained. Monismith, Ogawa, and Freeme[19] applied the hyperbolic rule to their test results, and found good comparison between predicted and actual curves. Deformations at lower stress levels could then be predicted.

Equations were developed to represent the relationship between applied stress and plastic strain at a particular number of stress repetitions, and these may be used to predict permanent deformation in fine-grained soils.

Monismith, Ogawa, and Freeme[19] also introduced the concept of cumulative loading in this type of material; i.e., of predicting the effect of cumulative loading in the field. There are two methods available to obtain the cumulative permanent strain from results of simple loading tests: a "time hardening" procedure, and a "strain hardening" procedure. These are illustrated in Figure 6.12, which is self-explanatory (total deformation after N_1 repetitions at σ_1 and N_2 repetitions at σ_2 being $\varepsilon_{p1} + \varepsilon_{p2}$). When these two approaches were used to predict the behavior of specimens tested with a combination of stress levels of 3, 5, and 10 psi, neither agreed quantitatively, but they were in qualitative agreement. The time hardening procedure provides better agreement if the stress levels are successively increased, while strain hardening is better when the loads are successively decreased. These two methods can, therefore, be used as a rough guide to bound the actual response.

In recent years, a method of controlling the magnitude of vertical compressive strain at the surface of the subgrade to some tolerable amount associated with a specific number of load repetitions has been proposed and adopted in some design procedures. By controlling the characteristics of the material in the pavement sections through proper design and construction procedures and by insuring that materials of adequate stiffness and sufficient thickness are used so that this strain level is not exceeded, rutting equal to or less than some prescribed amount can be assured. The use of limiting subgrade strain criteria developed for both highway and airfield pavements can be found in References 20-25, and the limiting subgrade strains adopted by a number of agencies are shown in Table 6.4.

Chou[26] at WES analyzed the accumulated rutting in the subgrade of flexible airfield pavements using laboratory repeated load tests. The pavements consisted of many full-scale test pavements trafficked and failed by multiple-wheel heavy gear loads and many pavements designed by the CBR equation. Based on the computed results of pavements designed for the same performance level using the CBR equation for different wheel loads and different subgrade moduli, Chou found that the accumulated permanent deformations at subgrade surface are not necessarily the same for each pavement but decrease with increasing subgrade modulus and increase with increasing wheel load, except the elastic vertical strain at subgrade surface which is nearly independent of these factors. Based on the results of the analysis, Chou concluded that the current concept of the control of subgrade rutting in flexible pavements is not strictly correct. Surface rut depth is not limited in many existing failure criteria proposed and used for flexible airfield pavements. In field tests conducted at WES, varying magnitudes of surface rut depth were measured in pavements which were considered to have failed at the same coverage levels, with larger rut depths measured in thicker pavements required for heavier loads. Computed results from field test pavements which were observed to have failed under accelerated traffic at the same coverage levels show that the elastic vertical strains at the surface of the subgrade are nearly the same but the magnitudes of subgrade rutting are quite different, with larger rutting computed for thicker pavements designed for heavier loads or for weaker subgrade conditions. Chou further concluded that unless surface rut depth is limited in the design procedure, subgrade rutting will not be controlled and if this is done, however, the limiting strain values proposed and used by many agencies will have to be modified to vary according to subgrade strength.

6.4 CONSTITUTIVE STRESS-STRAIN RELATIONS

6.4.1 RESILIENT STRESS-STRAIN

Extensive studies of the behavior of cohesive soils in laboratory repeated load tests were made by Seed and his associates at the University

of California at Berkeley. They found that the resilient modulus M_R did not depend on the confining pressure σ_3 but was sensitive to deviator stress σ_d. The relationship between M_R and σ_d generally has the shape shown in Figure 6.2. At low stress levels, M_R decreases rapidly with increasing values of σ_d; and, as σ_d further increases, there is only a slight increase in M_R. A bitumen material model, developed by Wang[27] in characterizing a highly plastic subgrade soil, has the following expression for M_R:

$$
\begin{aligned}
M_R &= K_1 + (K_2 - \sigma_d)K_3 \quad \text{for} \quad \sigma_d < K_2 \\
M_R &= K_1 + (\sigma_d - K_2)K_4 \quad \text{for} \quad \sigma_d > K_2
\end{aligned}
\qquad (6.1)
$$

Typical values for the resilient modulus of cohesive soils are summarized in Table 6.5.

The resilient modulus of cohesive soils was also found to be dependent on water content or suction. Data developed for undisturbed samples obtained from the San Diego Test Road[28] (Figures 6.13 and 6.14) illustrate the dependence of modulus on suction.

The variation of Poisson's ratio ν with stress is less clear, although Hicks and Finn[29] found that it remained constant or increased slightly with increasing repeated vertical stress. Poisson's ratio, however, appears not to be significantly affected by confining stress.

6.4.2 STATIC STRESS-STRAIN

A mathematical model characterizing the stress-strain relationship for soils was developed by Duncan and Chang.[30] The model is derived based on the assumption of a hyperbolic stress-strain relationship and the Mohr-Coulomb failure criteria. The tangent modulus E_t under any stress condition is expressed as

$$E_t = \left[1 - \frac{R_f(1 - \sin\phi)(\sigma_1 - \sigma_3)}{2c\cos\phi + \sigma_3 \sin\phi} \right]^2 E_1 \qquad (6.2)$$

where

$$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{unt}} = \frac{\text{principal stress difference at failure}}{\text{the maximum stress difference in the } \sigma_1 - \sigma_3 \sim \varepsilon \text{ curve}}$$

σ_1, σ_3 = the major and minor principal stresses

c, ϕ = Mohr-Coulomb strength parameters

E_1 = the initial tangent modulus from the $(\sigma_1 - \sigma_3) \sim \varepsilon$ curve

Equation 6.2 represents the nonlinear, stress-dependent, inelastic stress-strain behavior of soils and is convenient for use with the finite element method of analysis. The parameters c , ϕ , R_f , and E_i can be readily determined from results of standard laboratory triaxial tests.

6.4.3 PLASTIC STRESS-STRAIN

The methodology proposed by Barksdale[31] in predicting the plastic deformations for granular bases is also applicable to cohesive subgrade soils. The procedures are described in detail in Chapter 4 of this report.

The viscoelastic computer program (VESYS I) of the Federal Highway Administration[32] has the capability of computing vertical cumulative deflections of the surface of the pavement. Such calculations can be used as a measure of the pavement's ability to resist rutting damage. The amount of rutting occurring at the surface of the pavement will be reflected only in those material layers which are characterized as being viscoelastic. Layers which are assumed to be elastic contribute only to the resilient deformations of the pavement system.

6.4.4 DYNAMIC STRESS-STRAIN

The materials for the dynamic stress-strain relations presented in the granular material section are also applicable to cohesive soils, and are not repeated in this section. Of particular importance is the equation

$$E \text{ (in psi)} = 1500 \text{ CBR}$$
$$E \text{ (in kg/cm}^2) = 110 \text{ CBR}$$

which has been used extensively in characterizing cohesive subgrade soils, primarily because of lack of better and simpler procedures. Moreover, the wave propagation results obtained from the vibratory tests showed that the computed moduli of elasticity of the subgrade soil (4 CBR) were consistent when the vibrator was directly on the subgrade soil, but were erratic when the vibrator was placed on other component layers; the computed modulus was a function of the overburden pressures exhibited by the different pavement thicknesses.

6.4.5 SHEAR STRESS-STRAIN

The materials for the dynamic stress-strain relations presented in Chapter 4 are also applicable to cohesive soils, and are not repeated here.

6.4.6 MODULUS OF SOIL REACTION k

The support by the subgrade is the second major element in thickness design of concrete pavements. Subgrade (and subbase) support is estimated in terms of the Westergaard modulus of subgrade reaction k . It is equal to the load in pounds per square inch on a loaded area divided by the deflection in inches for that load, or the total load in pounds divided by the total volume displaced in cubic inches. k values are expressed as pounds per square inch per inch (psi/in.) or as pounds per cubic inch (pci). Where time and equipment are not available to determine k values, the relationships shown in Figure 6.15 are satisfactory for design purposes.

The load-deformation data obtained from plate bearing tests can be plotted in the form of a curve. The modulus of subgrade reaction k is the ratio of load in pounds per square inch to displacement of the bearing plate in inches. For example, if the load-deformation curve shows that a load of 7.5 psi results in a deflection of 0.05 in., k equals 7.5 divided by 0.05, or 150 pci or psi/in. The displacement of the bearing plate used in determining k should approximate the deflection of pavement slabs under expected wheel loads. The load-deformation

ratio at a displacement of 0.05 in. is generally used in determining k . However, the Corps of Engineers determines k for the deformation obtained under a 10-psi load.

In a design analysis, assumptions are made regarding the action of the subgrade or subbase-subgrade combination. Most concrete pavement designs have been based on the modulus of subgrade reaction k determined by load tests with a 30-in.-diam plate. This method treats the subgrade as though it had the load-carrying properties of a dense liquid. Influence charts developed by Pickett and Ray[33] are an extension of the Westergaard analysis and were developed for both the dense liquid subgrade assumption and the elastic solid subgrade assumption. The former has been used most frequently for pavement design.

The dense liquid subgrade assumption results in computed stresses that are somewhat higher than measured stresses. These differences are not great in most cases. The computed stress is on the conservative side and is suggested for design purposes.

6.5 EXPANSIVE SOILS

The following materials are taken from Packard.[34] Although Reference 34 is for rigid pavement, the materials are applicable to flexible pavement as well. Test methods to determine the expansive (high volume change) capacities of soils have been developed. The simpler tests provide indexes (such as plasticity index, shrinkage limit, and bar shrinkage) for identifying the approximate volume change potential of soils. For example, the following tabulation shows approximate expansion-plasticity relationships:

Plasticity Index (ASTM D 424)	Degree of Expansion	Approximate Percentage of Swell (ASTM D 1883)
0 to 10	Nonexpansive	2 or less
10 to 20	Moderately expansive	2 to 4
More than 20	Highly expansive	More than 4

Excessive differential shrink and swell of expansive soils in a subgrade create nonuniform support. As a result, the pavement placed on such a subgrade may become distorted and warped. Several conditions can lead to nonuniform support and damage to the pavement:

a. When expansive soils are compacted in too dry a condition or are allowed to dry out prior to paving, subsequent nonuniform expansion may cause pavement roughness.

b. When expansive soils are too wet prior to paving, subsequent nonuniform shrinkage may leave the slab edges unsupported or cause an objectionable increase in pavement crown.

c. When pavements are constructed over expansive soils with widely varying moisture contents, subsequent shrink and swell may cause bumps, depressions, or waves in the pavement surface. Similar waves may occur where there are abrupt changes in volume change capacities of subgrade soils.

Nonuniform support and pavement distortion from nonuniform shrink and swell of expansive soils are more likely to occur in arid, semiarid, or subhumid regions. Objectionable distortion can also occur in humid climates during periods of drought, during long dry periods in the summer months, or where subgrade soils are extremely expansive.

In all climatic areas, compaction of highly expansive soils when they are too dry can lead to detrimental expansion and softening of the subgrade during later rainy periods. The softening occurs more rapidly at joints and along pavement edges due to moisture infiltration. The resultant differential support may lead to pavement distress before the subgrade soils can adjust to the climatic environment and reach a more uniform and stable moisture content.

The following measures provide effective and economical control of expansive soils:

a. Subgrade grading operations. Selective grading, cross-hauling, and mixing of subgrade soils make it possible to have reasonably uniform conditions in the upper part of the subgrade, with gradual transitions between soils of varying volume change properties.

b. Compaction and moisture control. It is critically important to compact expansive soils 1 to 3 percent wet of AASHTO T 99 optimum moisture. Both research and experience show that expansive soils expand less on wetting and absorb less water

when compacted at this condition. After pavements are built, the moisture content of most subgrades increases to about the plastic limit of the soil (ASTM D 424); i.e., the moisture content reached is close to or slightly above the standard optimum. If this moisture content is obtained in construction, the subsequent changes in moisture will be much less and the subgrade will retain the reasonably uniform stability needed for good pavement performance.

c. Nonexpansive cover. In areas with prolonged periods of dry weather, highly expansive subgrades may require a cover layer of low volume change soil placed full width over the subgrade. This layer minimizes changes in the moisture content of the underlying expansive soil and also has some surcharge effect. If the low volume change layer has low to moderate permeability, it is not only more effective but usually less costly than a permeable, granular soil. Highly permeable, open-graded subbase materials are not recommended as cover for expansive soils since they permit greater changes in subgrade moisture content. Local experience with extremely expansive soils is the best guide for adequate depth of cover.

d. Cement-modified subgrade. The treatment of expansive clay soils with cement is very effective not only in reducing volume changes but in increasing the bearing strength of subgrade soils.

6.6 TESTING EQUIPMENT AND METHODS

Cohesive subgrade soils are nonhomogeneous, anisotropic, and nonlinear viscoelastic. Although they are not temperature-dependent as is bituminous concrete, they are highly dependent upon the rate of loading. Deacon[35] presented a detailed list of variables affecting general pavement material response; they are shown in Table 4.15 in Chapter 4.

In order to determine the elastic and viscoelastic properties of cohesive soils for use in a mechanistic pavement design and evaluation procedure, triaxial tests[36-39] and unconfined constant axial compression tests[32] have been used most frequently. The hollow cylinder simple shear test[40] is in the development stage at the University of Kentucky. The materials presented in Chapter 4 are also applicable to cohesive soils and are not repeated here.

The presentations and discussions on determining the resilient modulus and Poisson's ratio of general pavement materials using triaxial apparatus in Chapter 4 of this report are also applicable to cohesive soils. Creep tests designed to measure viscoelastic properties of cohesive soils are presented below.

The linear viscoelastic layer system computer program (VESYS I) developed at MIT under a Federal Highway Administration contract is capable of calculating the stresses and strains in a three-layer linear viscoelastic pavement system under static loads and accumulated permanent strains under repeated loads. In the material characterization, the linear viscoelastic creep compliance functions are determined from creep tests for bituminous concrete and cohesive subgrade soil, and the resilient modulus as determined from repeated triaxial tests is for unbound granular materials. The creep compliance is defined as the ratio of the time varying axial strain $\varepsilon(t)$ to the magnitude of the instantaneously applied axial stress S_o in an unconfined uniaxial compressive or tensile test as

$$J_t = \frac{\varepsilon(t)}{S_o} \tag{6.3}$$

Figure 6.16 shows a typical creep compliance curve. Curve fitting techniques are then used to input the compliance curves into the computer program. The creep tests were run for a period of 1000 seconds following two preliminary mechanical conditioning cycles.

An improved version of computer program VESYS I is under development at the University of Utah under contract to the Federal Highway Administration. The new program will have the capability of accounting for some of the nonlinear characteristics of pavement materials.

Table 6.1

Comparison of Complex and Resilient Moduli for Specimens of AASHTO Subgrade Soil

		Coffman, Kraft, and Tamayo,[10] Complex Modulus, psi $\sigma_d = 9.0 + 18.0$ psi, $\sigma_3 = 0$		Seed, Chan, and Lee,[8] Resilient Modulus, psi $\sigma_d = 10$ psi, $\sigma_3 = 3.5$ psi, Duration = 0.25 sec,
Percent	Dry Density pcf	0.16 Hz	16 Hz	Frequency = 20 Repetitions/Minute
13.5	114	4.5×10^3	6.2×10^3	13.0×10^3
15	114	2.9×10^3	4.9×10^3	8×10^3
16	114	0.5×10^3	4.5×10^3	6.6×10^3

Table 6.2

Summary of Regression Equations Including Soil Properties (from Reference 13)

Equation Number	a, intercept, ksi	b, Regression Coefficient: Percent Clay	PI	Percent Organic Carbon	Percent Silt	Group Index	Liquid Limit	R, Correlation Coefficient	$S_{\overline{X}}$, Standard Error of Estimate ksi
1	4.88	0.157						0.600**	2.76
2	5.12		0.235					0.581**	2.80
3	10.71			-2.14				-0.519**	2.95
4	15.59				-0.109	0.150		-0.450**	3.08
5	6.46							0.434**	3.10
6	4.32						0.114	0.330*	3.25
7	4.46	0.098	0.119					0.630**	2.70
8	6.90	0.0064	0.216	-1.97				0.757**	2.30
9	9.97	-0.0178	0.222	-1.88	-0.043			0.772**	2.26
10	6.37	0.034	0.450	-1.64	-0.0038	-0.244		0.796**	2.18
11	8.58	0.0586	0.1397		-0.0561			0.611**	2.68
12	3.63	0.1239	0.4792		0.0031	-0.3561		0.721**	2.49

* Significant @ $\alpha = 0.05$.
** Significant @ $\alpha = 0.01$.

Regression equation of the form:

$$E_{Ri} = a + b_1X_1 + b_2X_2 + b_nX_n$$

Table 6.3

Comparison Data for Determination of Resilient Modulus

Material Title	Classification	LL	PI	Field CBR percent	M_R by 1500 × CBR psi	Design Stress* psi	M_R @ Design Stress psi	Percent Error
Alum Creek	--	--	--	12	18,000	16.0	14,500	24
Loop I-220	CH	50	32	11.4	17,250	13.5	12,500	38
WES Loess	ML	27	3	6.8	10,200	12.0	11,000	7
Buckshot	CH	73	48	4.0	6,000	6.5	6,500	8
Burns Long Lake	CL	36	12	4.0	6,000	5.1	5,500	9
WES Poorhouse	CL	49	25	3.4	5,100	5.0	5,200	2
University of California**	--	--	--	3	4,500	4.7	4,700	4
MESL	CL	34	12	20	30,000	21.0	19,000	58

* From Figure C5 of Reference 16 for 6000 annual departures.
** From Reference 18.

Table 6.4

Limiting Subgrade Strain Criteria (from Table 8.9, Yoder and Witczak[16])

	Original Shell Oil Co.	Revised Shell Oil Co.	The Asphalt Institute	Kentucky Highway
Strain parameter	ε_{vs} [13]	ε_{vs}	ε_{vs}	ε_{vs}
Year introduced	1962-1965	1970-1972	1971-1973	1971-1973
Type pavement	Highways	Airfields	Airfields	Highways
Allowable strain				
$N_j = 10$	--	--	--	--
10^2	--	--	2548	--
10^3	2700	4500	1903	790
10^4	1680	2700	1646	639
10^5	1050	1700	1508	502
10^6	650	1030	1423	364
10^7	420	650	--	227
10^8	260	400	--	89
∞	--	--	1060	--
Effective E_1 (ksi)	140 (thin A.C.)	150	100	480 (33 percent A.C.)
	200 (thick A.C.)	--	--	300 (100 percent A.C.)

Note: ε_{vs} is maximum compressive subgrade strain ($\times 10^{-6}$ in./in.).

Table 6.5

Selected Measured Dynamic Moduli for Cohesive Subgrade Soil

Test Method	Material Description	Frequency/ Duration, cpm	Load Repetitions	Dynamic Modulus, psi
Repeated load triaxial [17]	Silty clay AASHTO Class A-6 w/c = 14 to 18% γ_d = 110 to 114 pcf	120 cpm 0.2 sec	10,000	w/c = 18%, E_R = 3-4,000 w/c = 16%, E_R = 7-8,000 w/c = 14%, E_R = 15-20,000
Repeated load triaxial [18]	Micaceous silty sand subgrade	33 cpm 0.1 sec	10,000	wet season E_R = 3-4,000 dry season E_R = 1.5-2,000
Repeated load triaxial [4]	Silty clay (AASHTO test) σ_d = 5 to 10 psi σ_3 < 3 psi γ_d = 110 to 115 pcf	20 cpm 0.25 sec		w/c = 13%, E_R = 13,000 w/c = 14%, E_R = 10,000 w/c = 15%, E_R = 8,000 w/c = 16%, E_R = 7,000 w/c = 17%, E_R = 2-5,000 w/c = 18%, E_R = 2,000
Repeated load triaxial [19]	Highly plastic clay (PI = 36.5) and silty clay (PI = 25.5)	30 cpm 0.1 sec	10,000	σ_d = 1.0, E_R = 4,150 σ_d = 5.2, E_R = 3,200
Repeated load triaxial [17]	Silty clay AASHTO Class A-7-6 w/c = 11 to 20% γ_d = 102 to 105 pcf	120 cpm 0.2 sec	10,000	w/c = 20%, E_R = 7-10,000 w/c = 18%, E_R = 15-16,000 w/c = 16%, E_R = 14-15,000
Repeated load triaxial [14]	Silty clay AASHTO Class A-6 to A-7-6	30 cpm 0.1 sec	100	E_R = 10,000 (1 atms) [1] E_R = 100,000 (10 atms)

Notes:

[1] Refers to soil moisture section at time of test.

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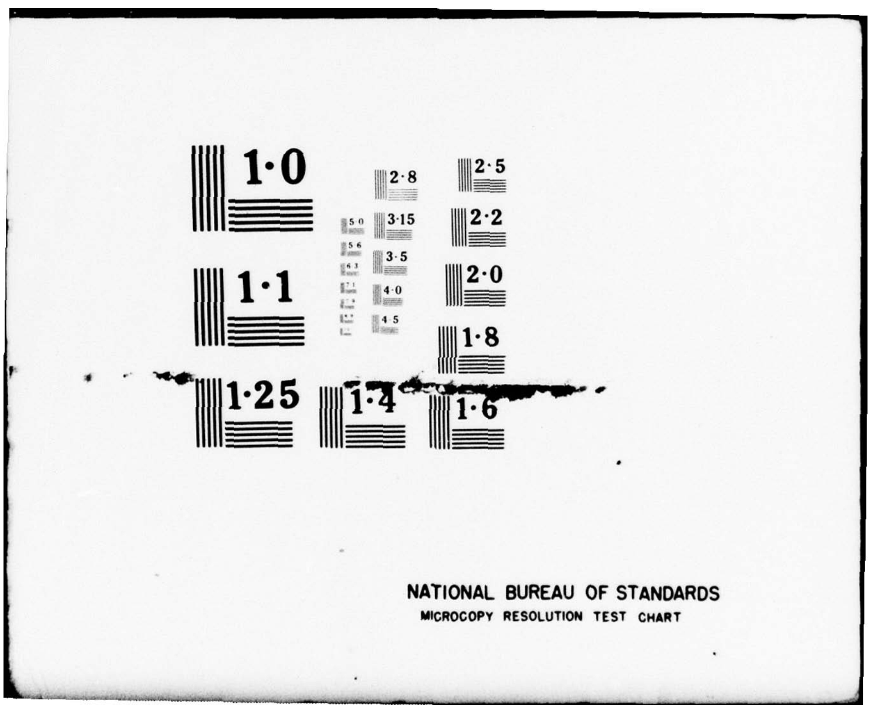
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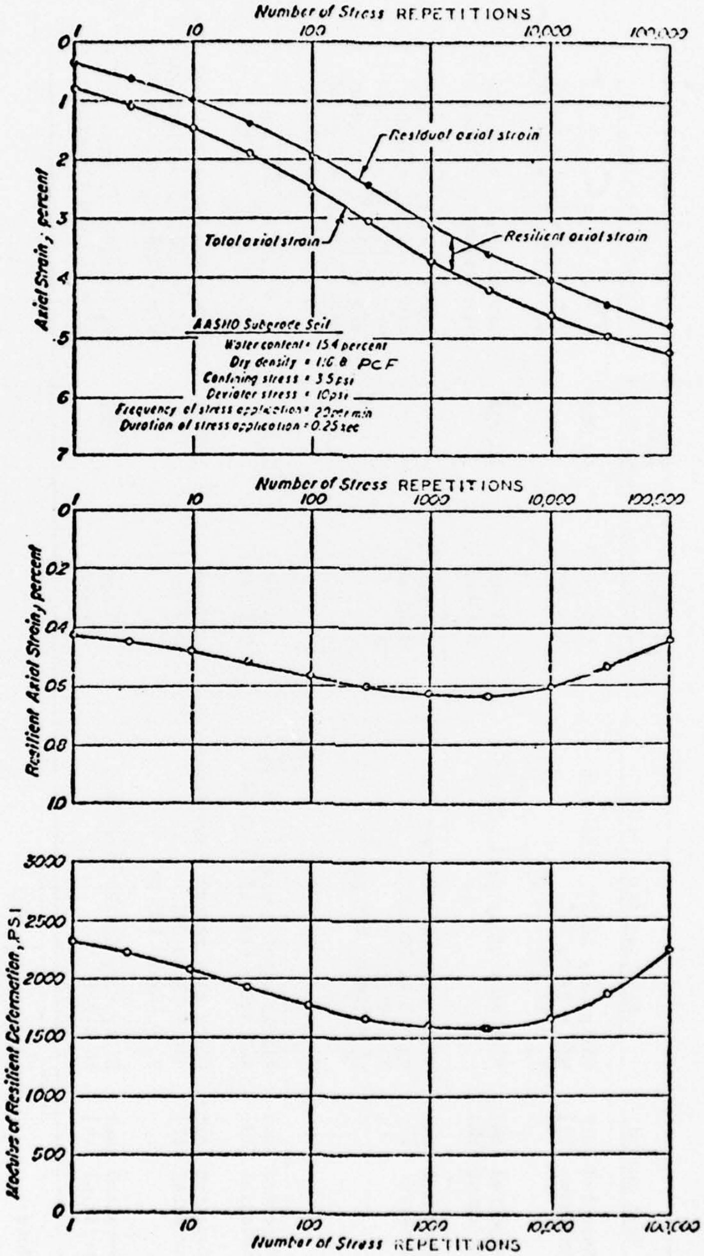


Figure 6.1. Typical results of repeated loading triaxial compression test (after Seed, Chan, and Lee[8])

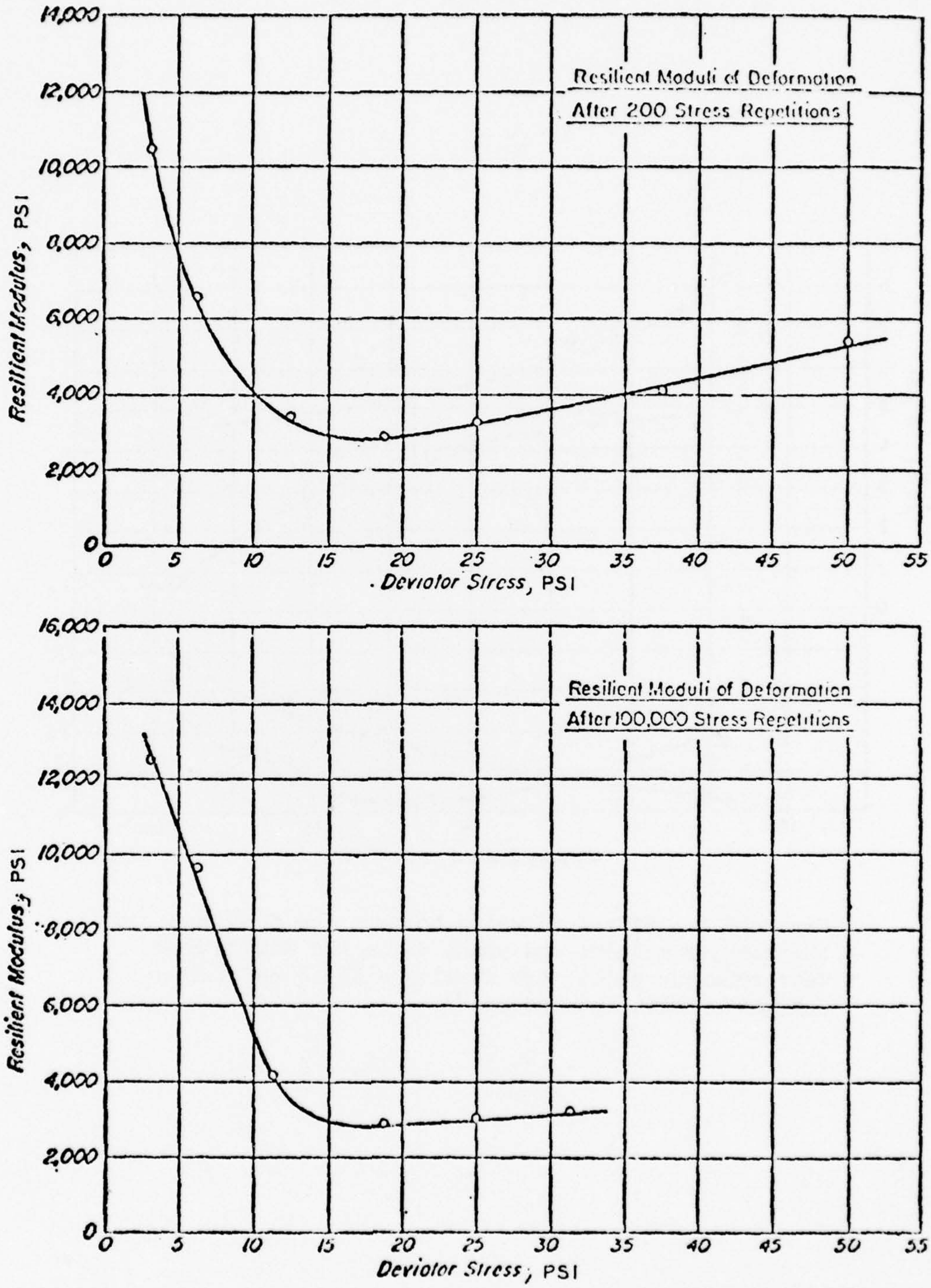


Figure 6.2. Effect of stress intensity on resilience characteristics; AASHTO Road Test subgrade soil (after Seed, Chan, and Lee[8])

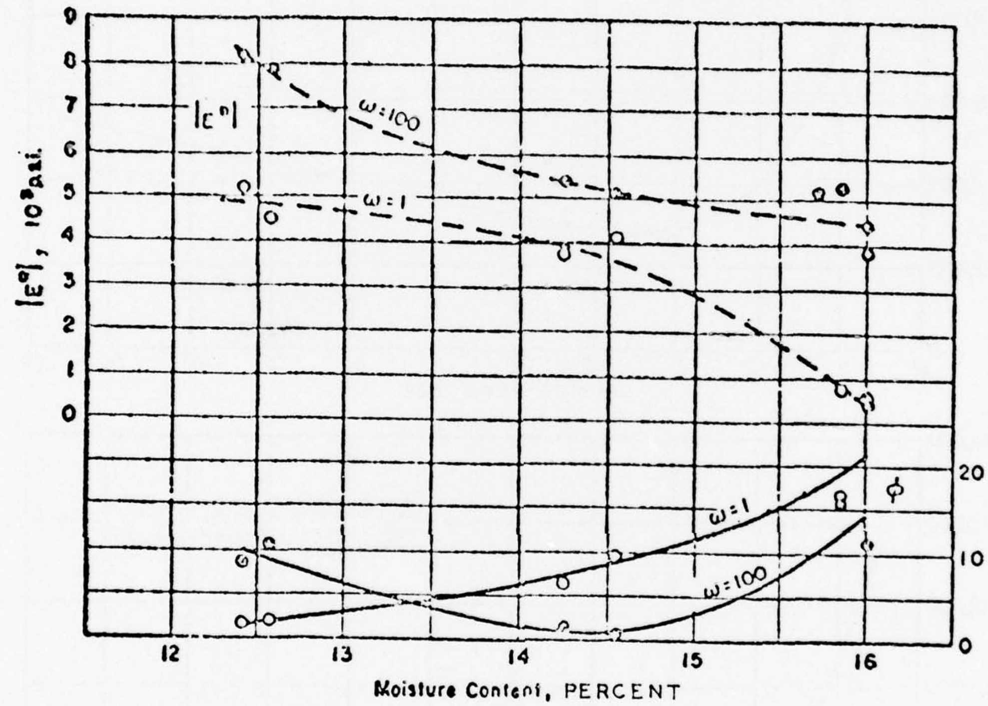


Figure 6.3. Effect of water content and frequency on the complex modulus and phase angle for AASHTO Road Test subgrade soils; dry density = 113.8 pcf (after Coffman, Kraft, and Tomayo[10])

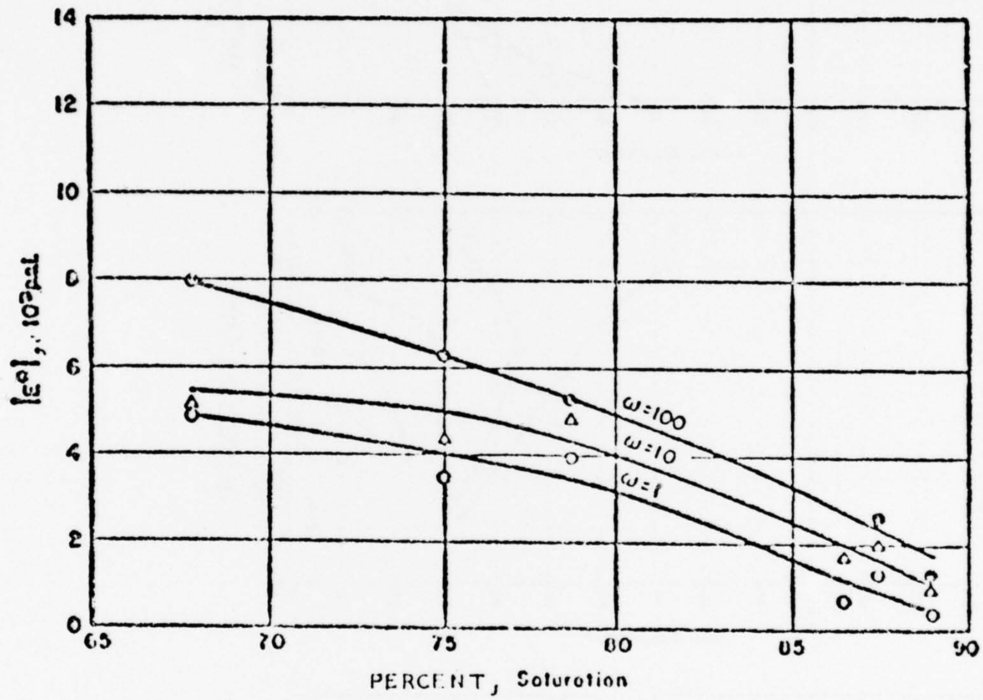
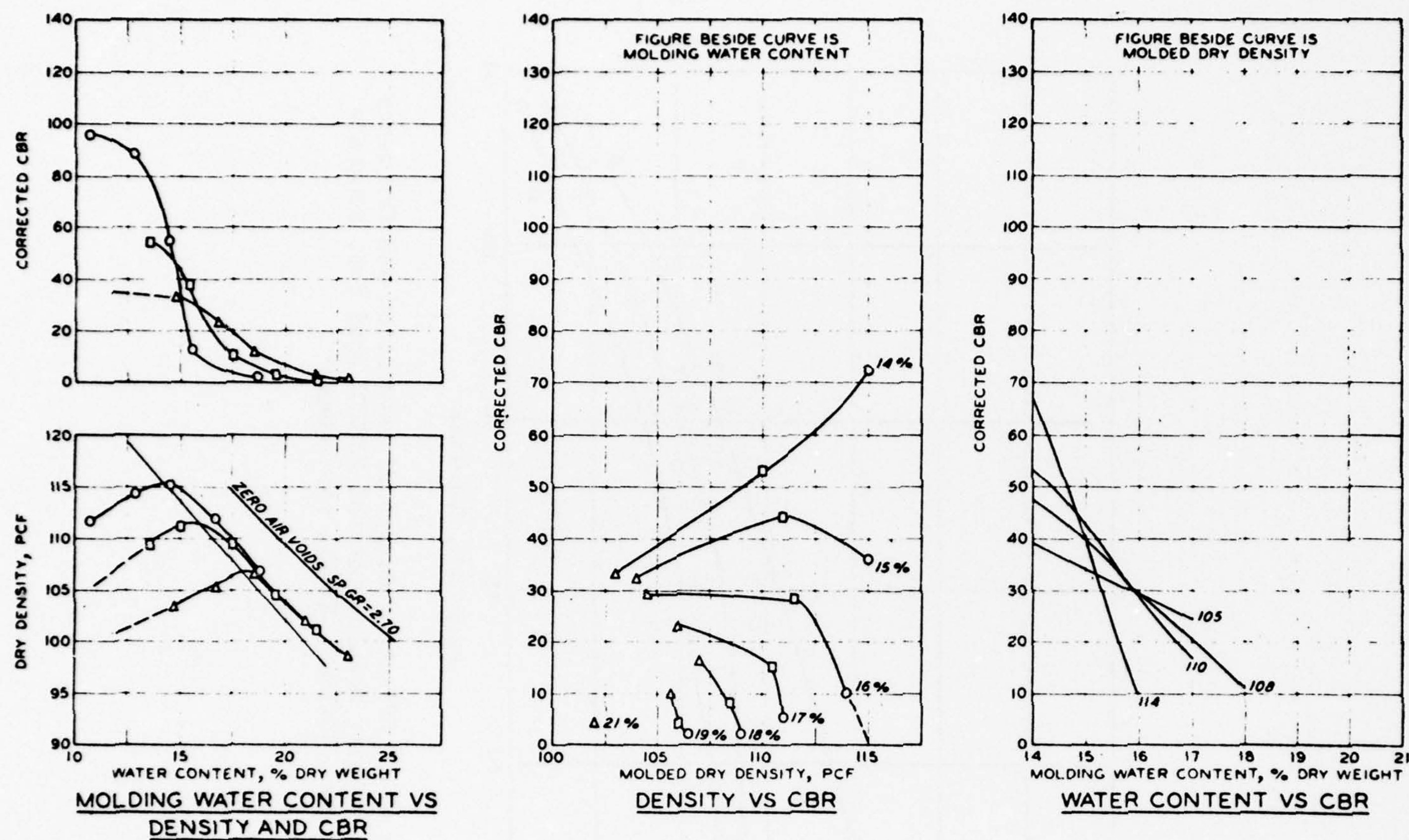


Figure 6.4. Effect of degree of saturation and frequency on the complex modulus for the AASHTO Road Test subgrade soil (after Coffman, Kraft, and Tomayo[10])



LEGEND

	NO. OF BLOWS/ LAYER	COMPACTION EFFORT
Δ	12	CE 12
□	26	CE 26
○	55	CE 55

Figure 6.5. CBR, density, and water content data for lean clay subgrade material (tested as molded) (from Reference 17)

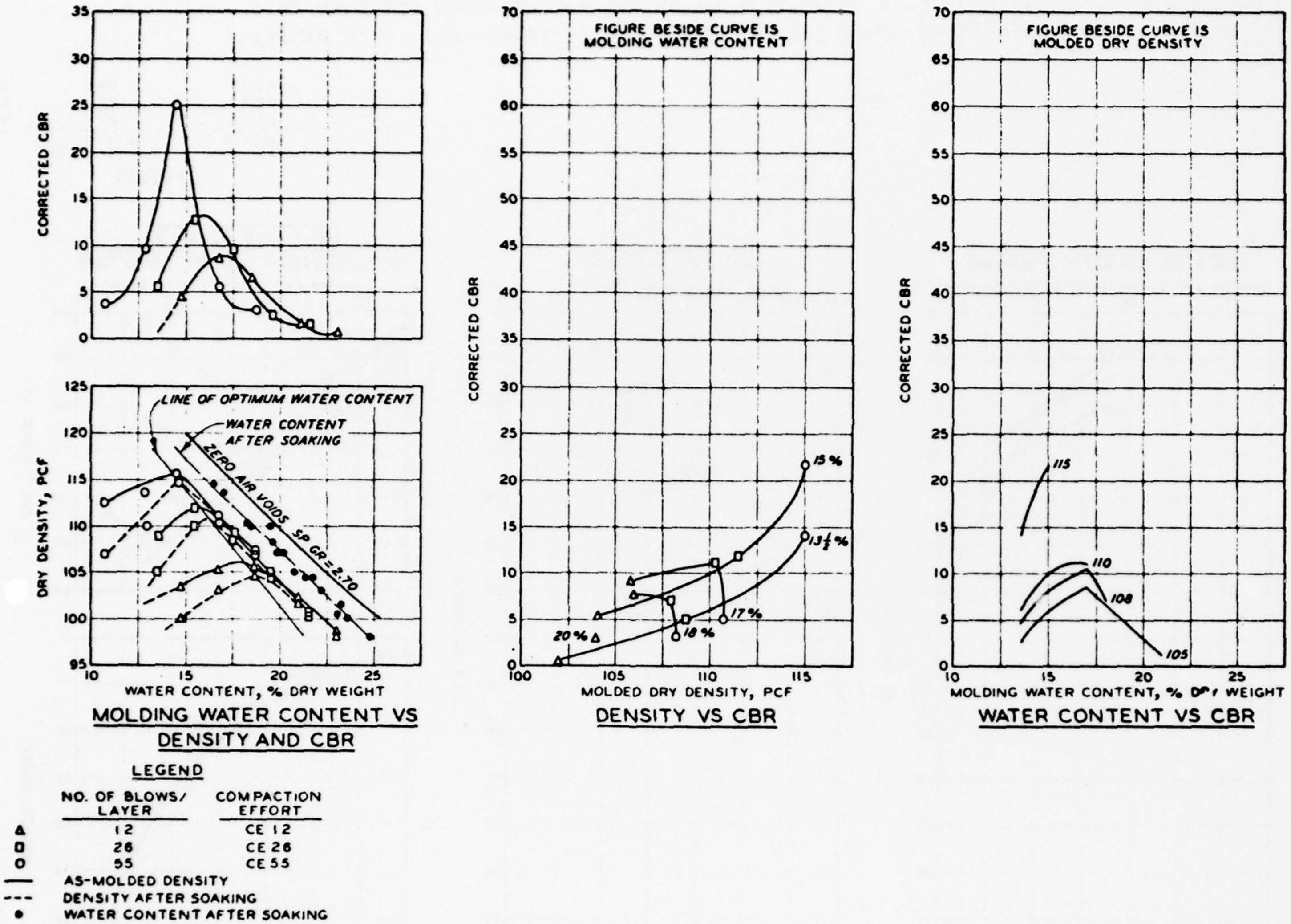
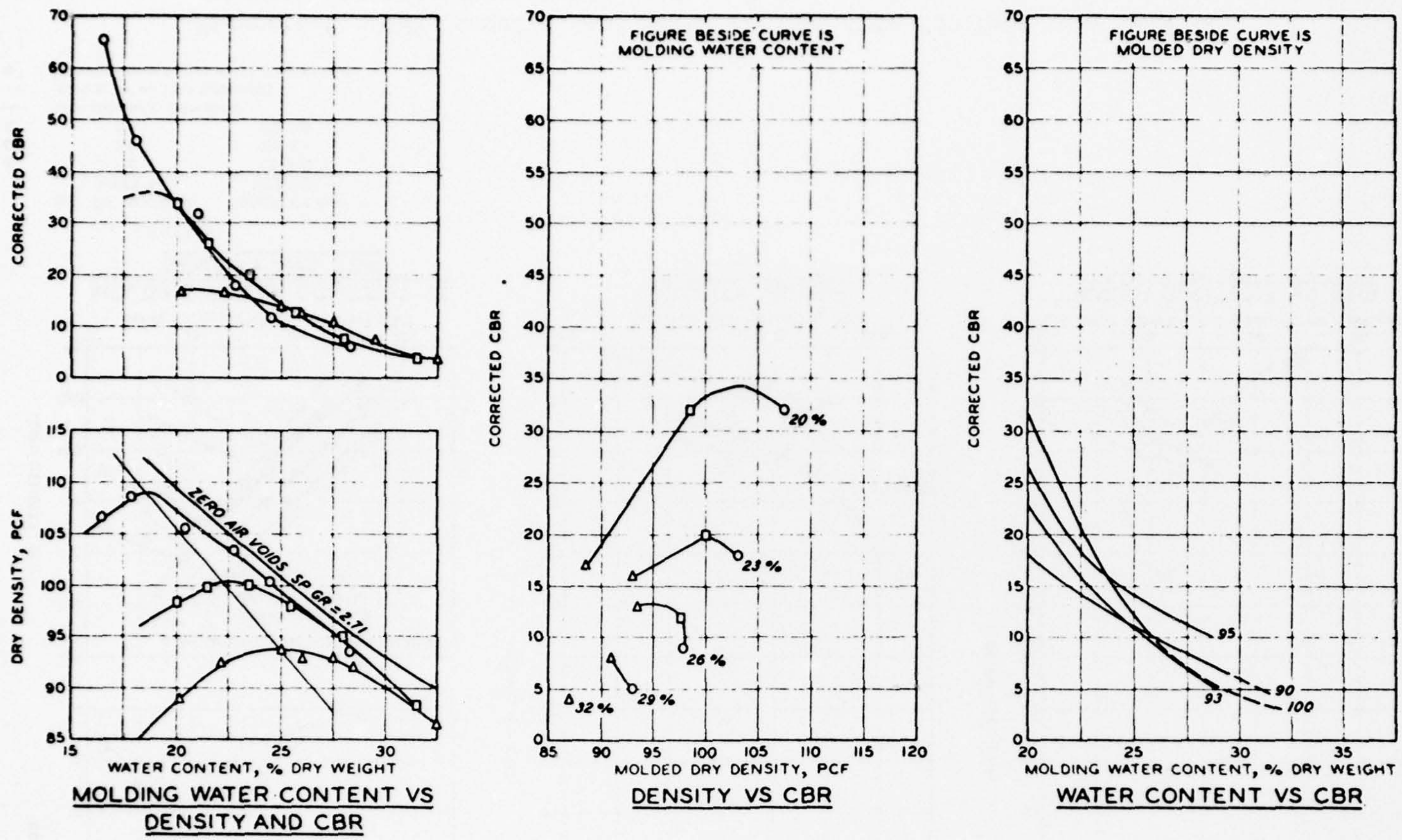


Figure 6.6. CBR, density, and water content data for lean clay subgrade material (tested after soaking) (from Reference 15)



LEGEND

	NO. OF BLOWS/ LAYER	COMPACTION EFFORT
Δ	12	CE 12
□	26	CE 26
○	55	CE 55

Figure 6.7. CBR, density, and water content data for heavy clay subgrade material (tested as molded) (from Reference 15)

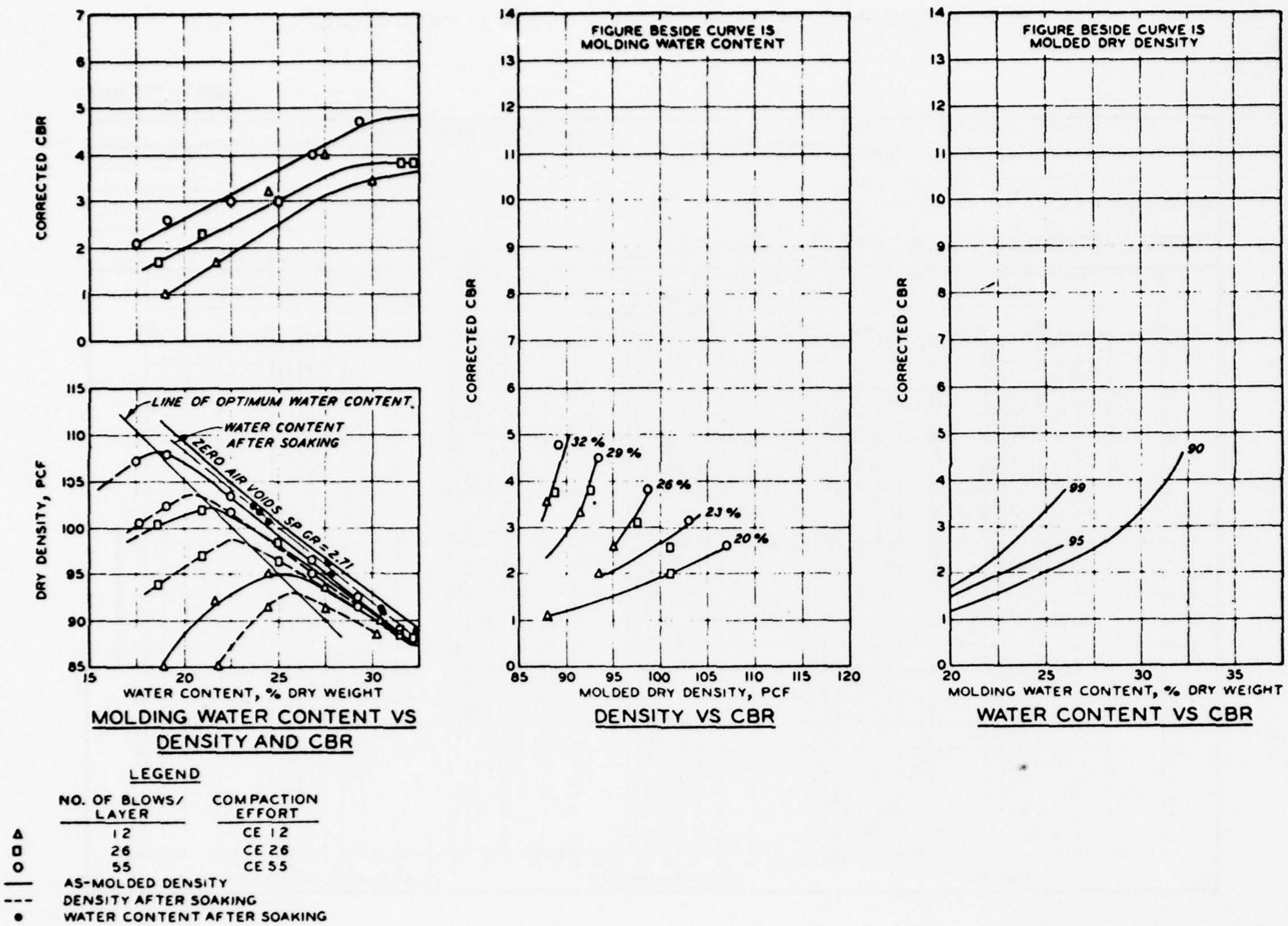


Figure 6.8. CBR, density, and water content data for heavy clay subgrade material (tested after soaking) (from Reference 15)

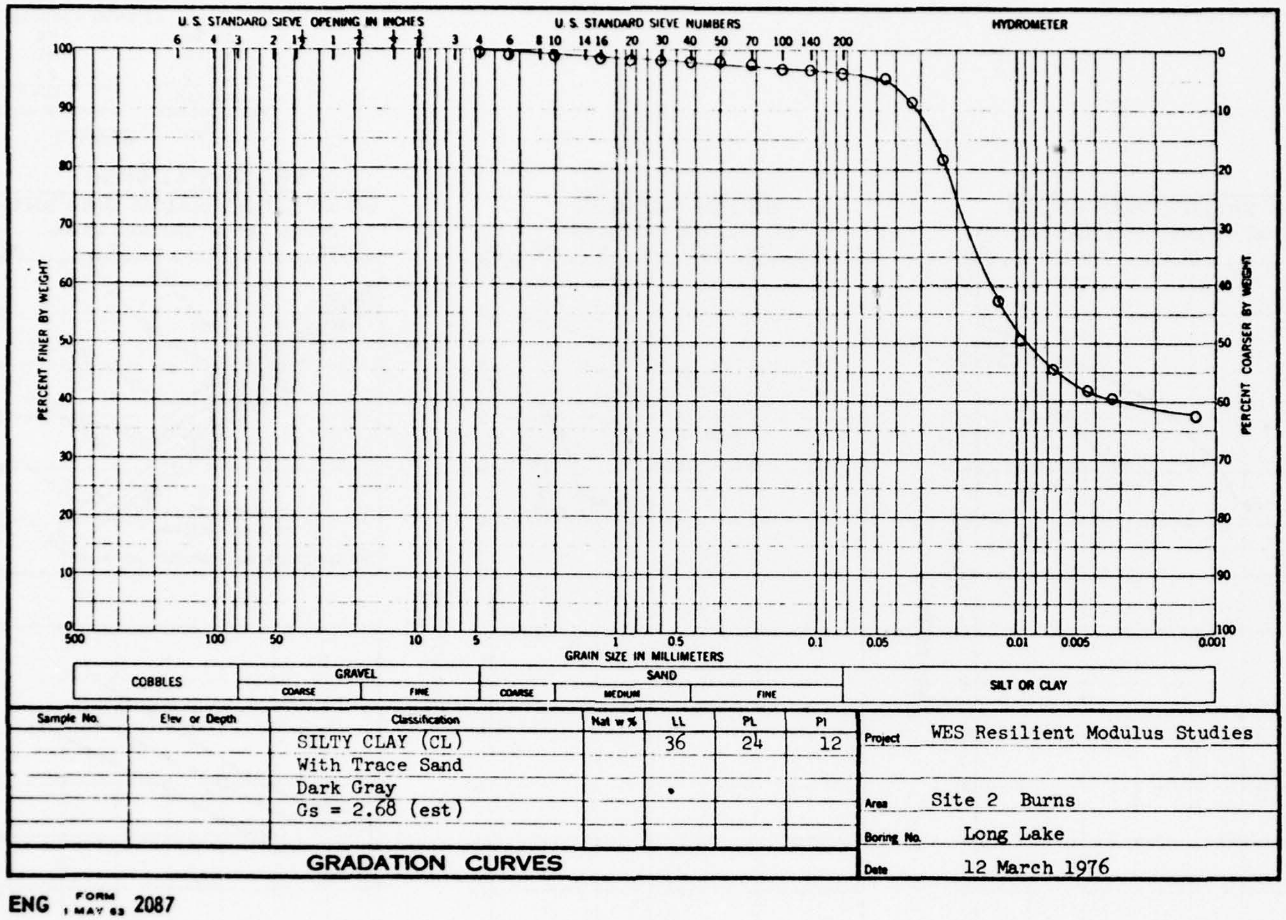


Figure 6.9. Gradation curve for Burns Long Lake sample

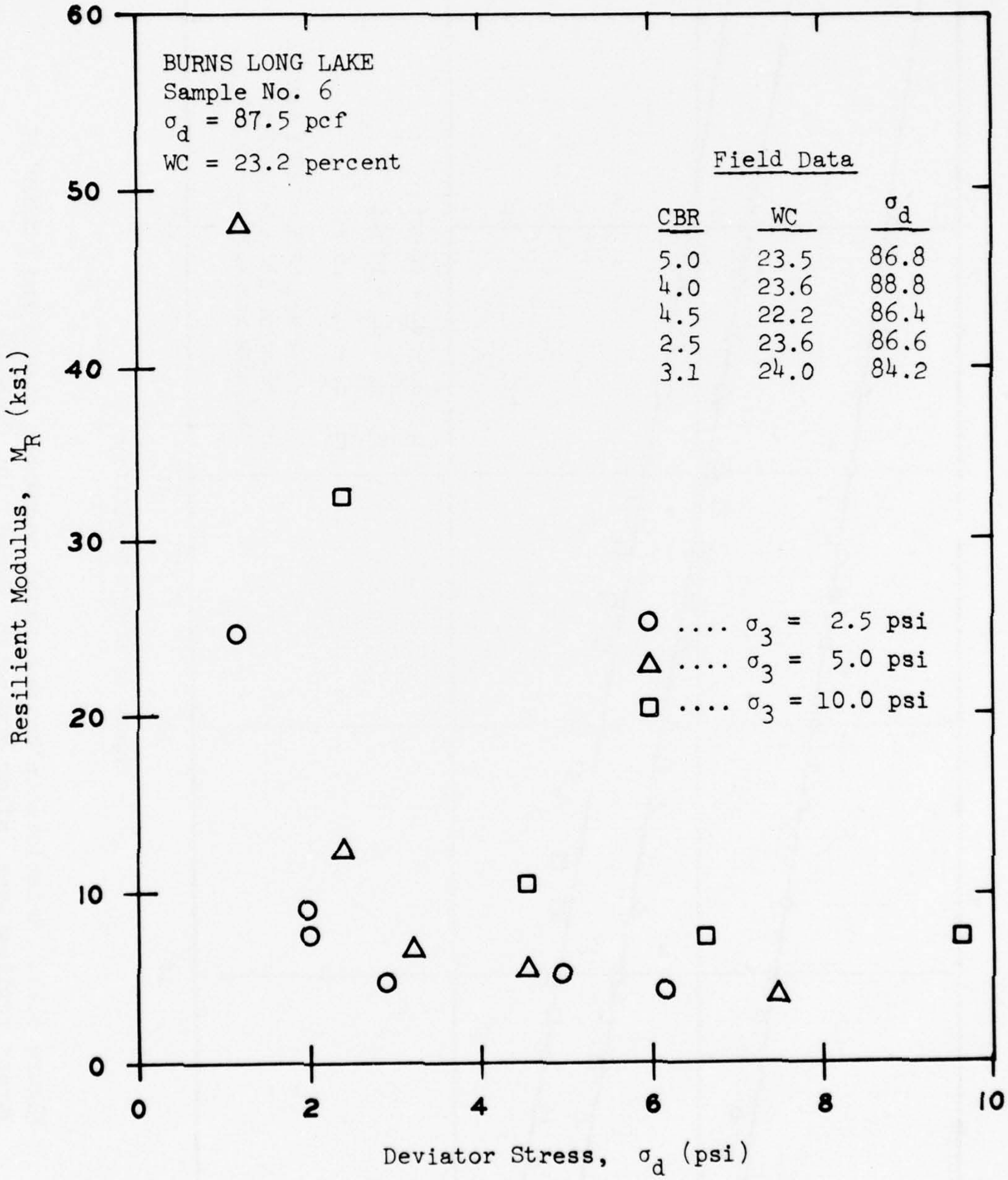


Figure 6.10. WES test data for Burns Long Lake sample

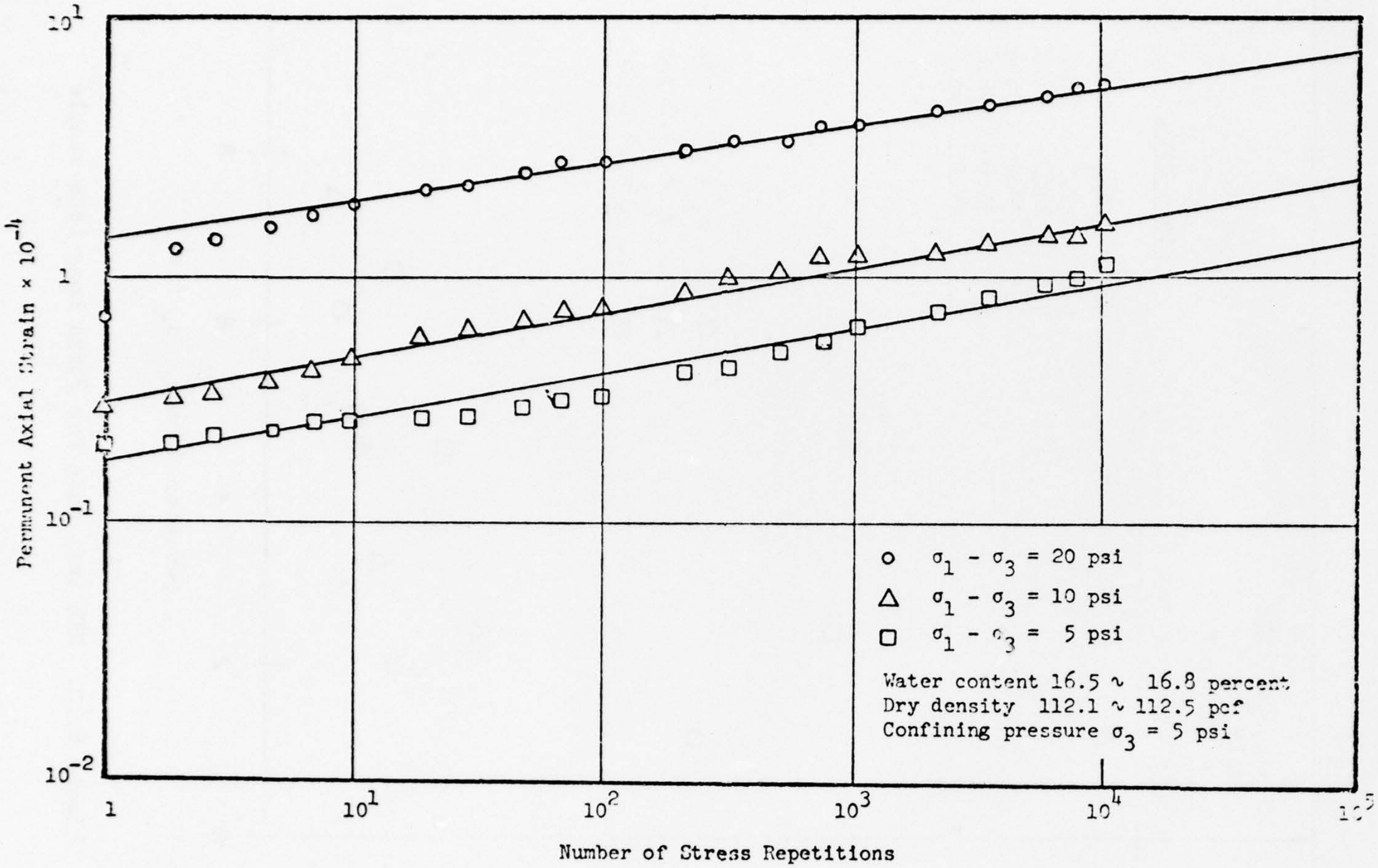


Figure 6.11. Relationship between axial permanent strain and number of stress applications (after Monismith, Ogawa, and Freeme[19])

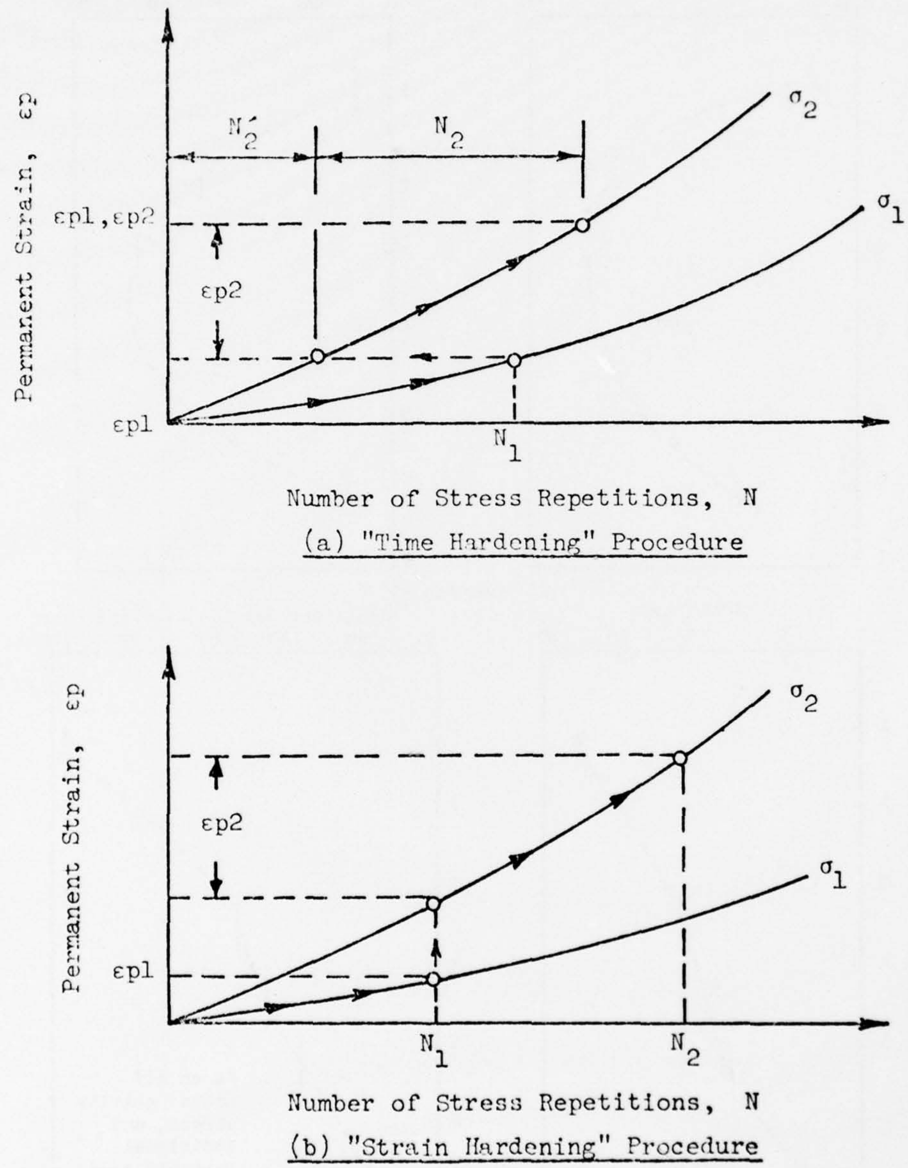
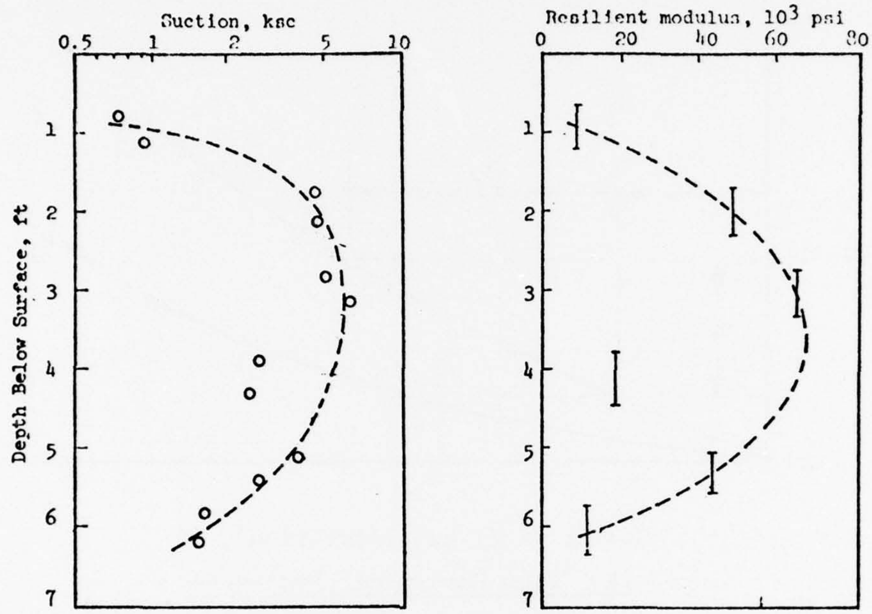
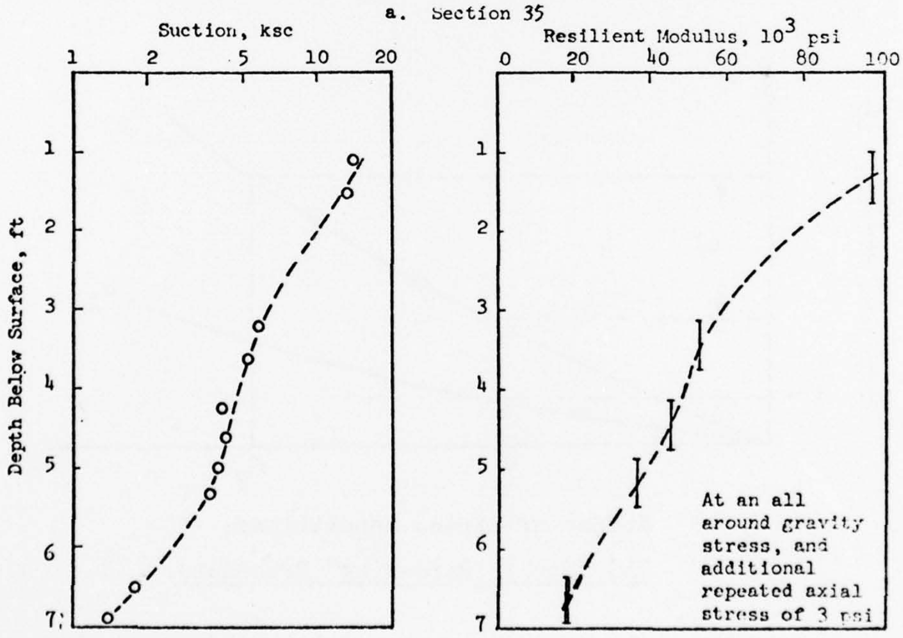


Figure 6.12. Procedures to predict cumulative loading from the results of simple loading tests (after Monismith, Ogawa, and Freeme[19])

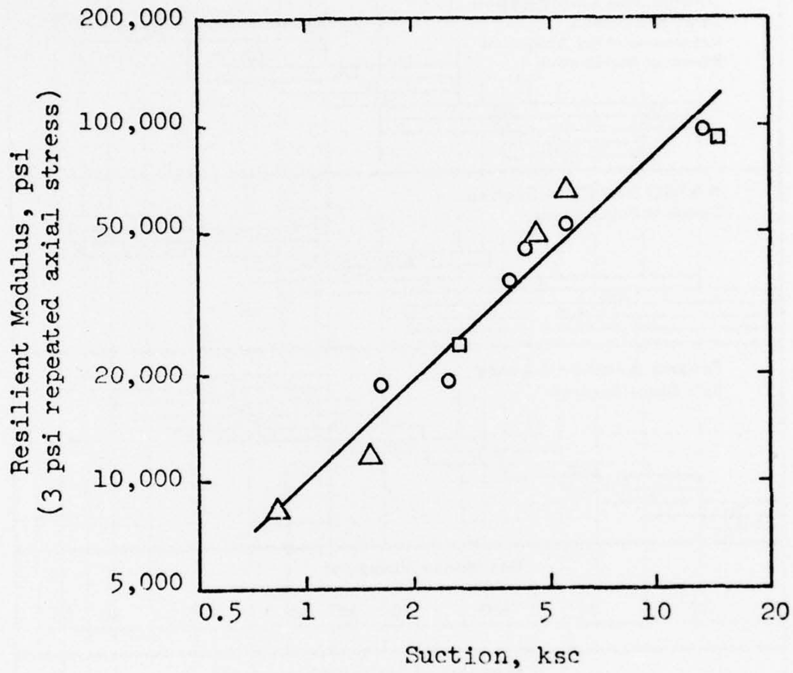


a. Section 35



b. Section 2

Figure 6.13. Suction and resilient modulus (after Dehlen[28])



o Undisturbed samples, Section 2

□ Undisturbed samples, Section 35

△ Laboratory compacted samples

Figure 6.14. Relation between resilient modulus and suction, San Diego Road Test subgrade soil (after Dehlen[28])

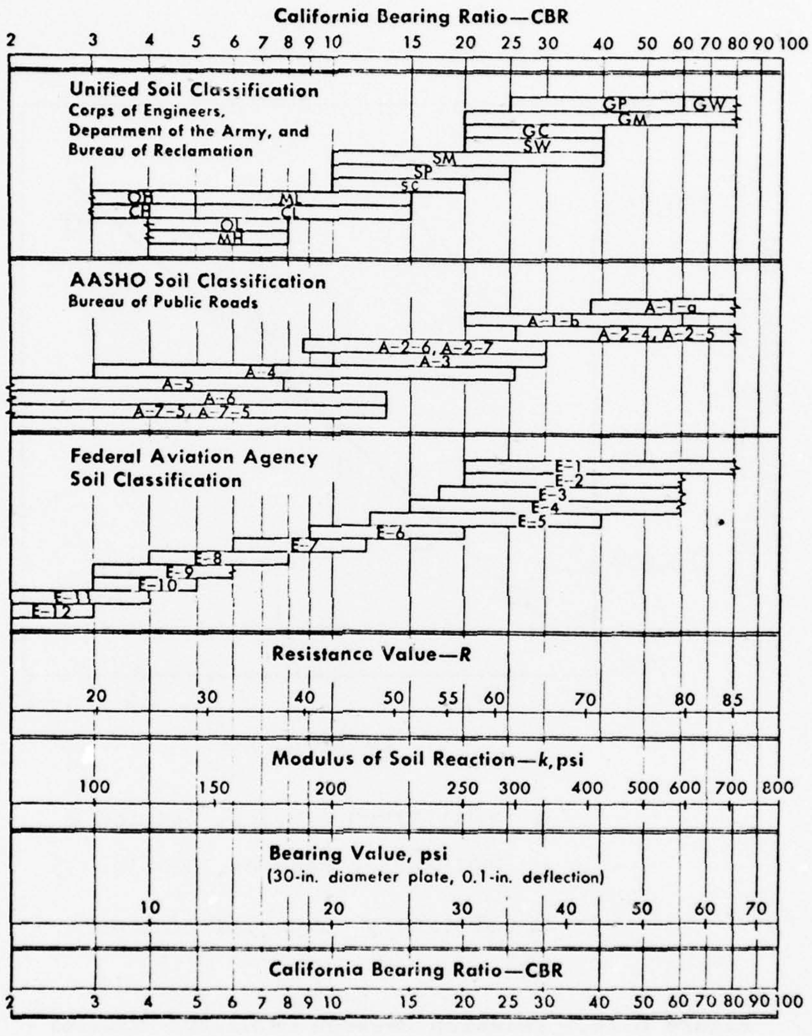


Figure 6.15. Approximate interrelationships of soil classification and bearing values

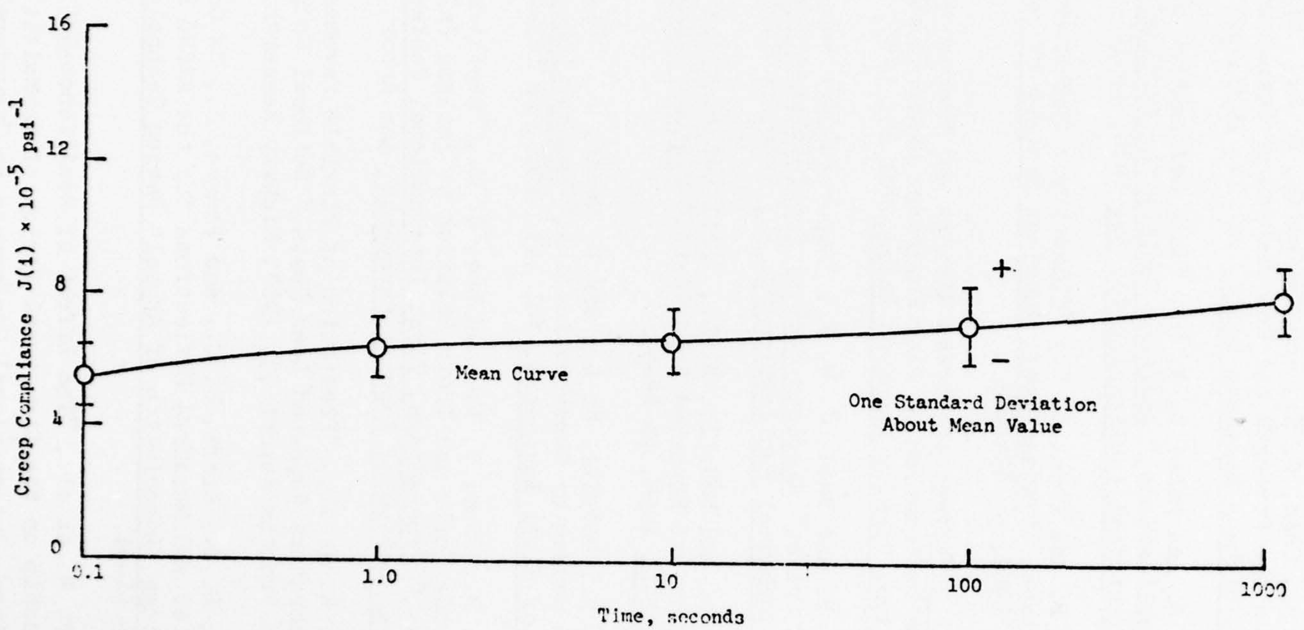


Figure 6.16. Creep compliance of clay, short-term tests

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NOTATION

c	Cohesion
E	Modulus of elasticity
E^*	Complex modulus
E_i	Initial tangent modulus
E_R	Dynamic modulus
E_t	Tangent modulus
E_1	Modulus
J_t	Creep compliance
k	Westergaard modulus of soil reaction
K_1, K_2, K_3, K_4	Coefficients
M_R	Resilient modulus
N	Number of load repetitions
N_j	Number of load repetitions to failure
PI	Plasticity index
R	Resistance value
R_f	$\dfrac{\text{Principal stress difference at failure}}{\text{Maximum stress difference}}$
S_o	Instantaneously applied axial stress
w/c	Water content
γ_d	Dry density
ε	Strain
ε_p	Permanent strain
$\varepsilon(t)$	Time varying axial strain
ε_{vs}	Maximum compressive subgrade strain
ν	Poisson's ratio
σ	Stress
σ_d	Deviator stress
σ_1	Major principal stress
σ_3	Confining pressure; also, minor principal stress
$\left(\sigma_1 - \sigma_3\right)_f$	Principal stress difference at failure

$(\sigma_1 - \sigma_3)_{ult}$	Maximum stress difference
ϕ	Angle of internal friction
ω	Frequency

CHAPTER 7: SUMMARY AND RECOMMENDATIONS FOR FUTURE WORK

7.1 GENERAL

This report presents the basic engineering properties of pavement materials with respect to highway and aircraft loadings and environmental conditions, and summarizes the important works conducted by the researchers. The materials covered are bituminous mixtures, portland cement concrete, granular materials, chemically stabilized soils, and fine-grained soils.

Since the advent of high-speed digital computers, much research effort has been directed toward development of mathematical models to compute pavement responses to traffic loadings. The response is generally critical stresses and strains in the pavement, and the mathematical models include Burmister's layered elastic solution, viscoelastic layered analysis, finite element and finite difference techniques, slab on elastic solid or on liquid foundation, and many others. More research effort has been devoted to studying traffic-associated phenomena than nontraffic-associated. Because the success of a mechanistic model lies in the correctness of the stress-strain relationships of pavement materials which are input into the programs to compute the pavement response, effort has been focused upon the measurements of modulus (or stiffness) and Poisson's ratio of the materials in the laboratory. In recent years, research effort has concentrated on the study of permanent deformation characteristics of materials to minimize the amount of surface rutting in flexible pavements. The mathematical models developed are considered to be superior to the old design methods, which are mostly empirical in nature.

The essential purpose of the newly developed computer-oriented mathematical models is to relate computed stresses and strains to pavement performance. Miner's theory of cumulative damage[1] is one of many methodologies used for this purpose. The procedure requires the computations of the cumulative damage at different locations in the pavement

caused by each design aircraft type for design operation level and environment ranges through the design service life. The aircraft can have different gear configurations and wander characteristics. The theory has gained much popularity and acceptance in recent years and is an advance in design concept because conventional design methods consider only a critical or design aircraft load which is static and does not wander across the pavement.

It is the author's opinion that if a mechanistic approach is used to compute pavement response to traffic loadings, the cumulative damage theory based on Miner's hypothesis provides an excellent procedure for prediction of pavement performance, provided that correct failure criteria (usually strain-load repetition relationships) which truly represent pavement behavior in the field are used. Otherwise, the added complex and lengthy computation would not increase the accuracy and reliability of the results.

Much research effort has been devoted to development of failure criteria for different distress modes. In general, subgrade failure criteria were established based on existing design curves or field performance data, and fatigue failure criteria were established from laboratory repeated-load tests. In using the subgrade failure criteria, it should be noted that not all field test pavements from which the design curves were drawn were failed exactly in the subgrade soil. In using the fatigue failure criteria, it should be noted that failure defined in the laboratory is not always compatible with failure defined for the in-service pavement. To the author's knowledge, no research effort done has been directed toward correlation of laboratory-determined failure criteria and in-service pavement life (i.e., repetitions to failure for a given strain level). Brown and Pell[2] suggested that in-service pavement life is of the order of twenty times the life of a test specimen in the laboratory. Witczak[3] analyzed pavements from the Baltimore-Washington International Airport and compared damage between Kingham and Monismith criteria. He found that completely different patterns exist in the relationship between damage and bituminous concrete modulus (see Figure 15a

of Reference 3). Monismith criteria were established based on laboratory repeated-load tests, and Kingham criteria were derived from AASHTO road test data based on functional failure conditions (terminal serviceability level). The obvious disparity between the results computed using different failure criteria strongly suggests the urgent need of research to verify, by means of field performance studies, the applicability of failure criteria developed from laboratory results.

The results of this study have shown that (a) there are many different types of testing equipment and procedures to quantitatively measure the resilient and rutting properties of pavement materials, and (b) these properties in turn vary considerably with testing procedure and equipment. Some of the main variables which affect the results are sample size, load frequency, rest period, and temperature. It is the author's opinion that to develop a universally acceptable mathematical model for predicting pavement response to loads, standardized universally accepted testing procedures to measure material constants must first be developed and practiced. Research effort with high priority should be initiated in this area.

7.2 SPECIFIC

7.2.1 BITUMINOUS MIXTURES

Extensive research effort has been made investigating the rheological and fatigue properties of bituminous mixtures. A bituminous mixture is thermoviscoelastic in nature; its response to load is dependent upon rate of loading and temperature. Pure bitumen exhibits basically linear engineering behavior, but starts to exhibit nonlinear load-deformation characteristics as aggregate is added to the mix. It is the author's opinion that there is no need to conduct any further extensive laboratory fatigue tests on bituminous concrete. Future research should emphasize the verification of laboratory results and theoretical analysis by means of field performance data. The results of this study have demonstrated that the fatigue response is considerably variable (e.g., see Figure 2.22). To have a reasonable and reliable fatigue

design subsystem, this variable should be incorporated into the subsystem. Therefore, a research effort is needed to develop a probabilistic fatigue design subsystem.

The prediction of permanent deformation of bituminous concrete is at its infant stage. However, controversial concepts and different results have already been presented by different agencies. Evidently, the layered elastic analysis incorporated with laboratory repeated-load tests presents certain difficulty in predicting permanent deformations. A different approach should therefore be attempted, such as the theory of nonlinear viscoelasticity.

7.2.2 PORTLAND CEMENT CONCRETE

The state of the art for PCC is more advanced than that for materials used in flexible pavements. If it is desired to predict pavement performance utilizing cumulative damage theory based on Miner's cumulative damage theory, some research effort should be devoted to revalidating the existing fatigue curves and possibly improving them. Research effort can also be devoted to further evaluate the values of modulus and Poisson's ratio of PCC which are input into the mechanistic model to compute the flexural stress in the concrete slab. The modulus of cracked section can also be investigated.

7.2.3 GRANULAR MATERIALS

Based upon the extensive studies presented in this report, it is the opinion of the author that there is no need to conduct any further extensive laboratory tests studying the basic elastic and plastic properties of untreated granular materials in relation to variables such as loadings, moisture, percentage of fines, and others, because an abundance of such information is available. Recent research effort has been concentrated on developing stress-strain relations in connection with the use of mechanistic models to compute pavement response to loadings. Results presented by many researchers have demonstrated that the concept of stress-dependent modulus developed at the University of California at Berkeley is workable. The stresses and displacements computed by means

of stress-dependent moduli were much closer to the measured values than those computed by means of stress-independent moduli.

When the layered elastic program is used to compute stresses and displacements in a pavement structure, tensile radial stresses exist at the bottom layers of the granular materials. Comparisons between computed and measured values have indicated that the presence of tensile stress does not seem to affect the accuracy of the computed vertical stresses and displacements, but they posed a serious problem in the use of laboratory repeated-load test data to compute permanent deformations in the pavement. The problem lies in the fact that compressive confining pressures are always used during the test to prevent the specimen from collapsing under the load applications. In other words, untreated granular materials cannot resist tensile stresses but the computer program predicts that tensile stresses occur at the bottom of granular layers. It can be concluded that (a) the states of stress existing in the granular layers under aircraft loadings are extremely complicated and cannot be simply described by constant values of vertical compressive stress σ_1 and horizontal stress σ_3 which are computed by the layered elastic program; and (b) the response of the granular materials to the repeated applications of aircraft loads cannot be correctly simulated by the laboratory repeated-load triaxial tests.

It is the author's opinion that extensive research effort should be directed toward development of a procedure which can predict permanent deformation in the untreated granular layers. A computer program should be developed which can correctly describe the states of stress existing in the granular layers under aircraft loadings, and a laboratory test procedure should be developed which can correctly simulate the response of the granular materials to aircraft loadings which are repetitive in nature and wander across the pavement.

7.2.4 SOIL STABILIZATION

Information on basic engineering properties of stabilized soil is abundant. Shrinkage cracking has always been a problem in cement-treated materials since it induces cracking in overlying layers. Research should be conducted to develop additives to reduce such cracking without

severely reducing the strength of the mixture. Considerable research effort should also be focused upon the study of resilient properties of soil-cement and bituminous-stabilized soils subjected to aircraft loadings. Because of the susceptibility of cement-treated materials to cracking, some engineers assume that cement-treated materials cannot resist tensile stress and a modulus comparable to that of an untreated granular material is used in the analysis. It is felt that more verification of such an approach is warranted because modulus of a soil-cement specimen measured in the laboratory is different from that of untreated granular material.

Research in fatigue behavior of stabilized soil is practically nonexistent. Since soil-cement is mostly placed in upper layers subjected to high stresses from moving aircraft loads, research is needed to assess the fatigue behavior of this material and the influence of mix variables upon its fatigue life.

7.2.5 FINE-GRAINED SOILS

Based on the extensive studies presented by many researchers, it is the author's opinion that there is no need to conduct any further extensive laboratory tests to evaluate the basic engineering properties of fine-grained soils. Pioneer works conducted at MIT, the University of California at Berkeley, WES, and many other institutions have thoroughly investigated the strength characteristics of fine-grained soil in relation to variables such as compaction effort, moisture content, and load frequency. Recent investigations on resilient properties of fine-grained soils have indicated that the resilient modulus can be related to moisture content or suction capability. Very little research has been done investigating the characteristics of permanent deformation in fine-grained soils and there is much need for research in this area. Since the question arose that limiting subgrade vertical strain does not guarantee the control of permanent deformation in subgrade soil, research effort should be focused upon this area to search for a better solution.

Miner's cumulative damage theory in fatigue has been used successfully in studying the fatigue property of bituminous concrete materials.

Witczak did the pioneer work in establishing a failure criterion for subgrade soil based on Corps of Engineers' field test data. He then developed the design criteria for full-depth bituminous concrete airfield pavements using Miner's cumulative damage theory. It is the opinion of the author that it is acceptable to apply Miner's damage theory in fatigue to subgrade soil, which is generally failed by shear deformation. However, a minimal amount of research effort should be devoted to either confirming or invalidating the practice.

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